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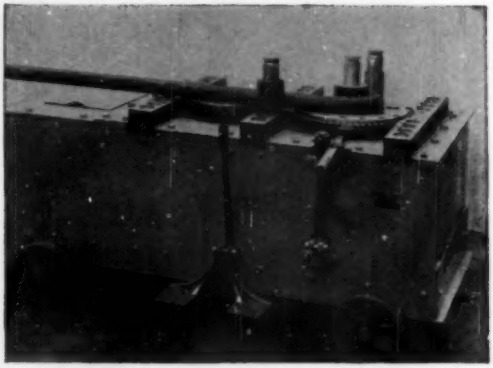

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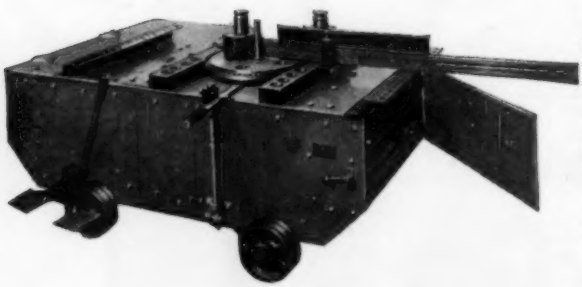

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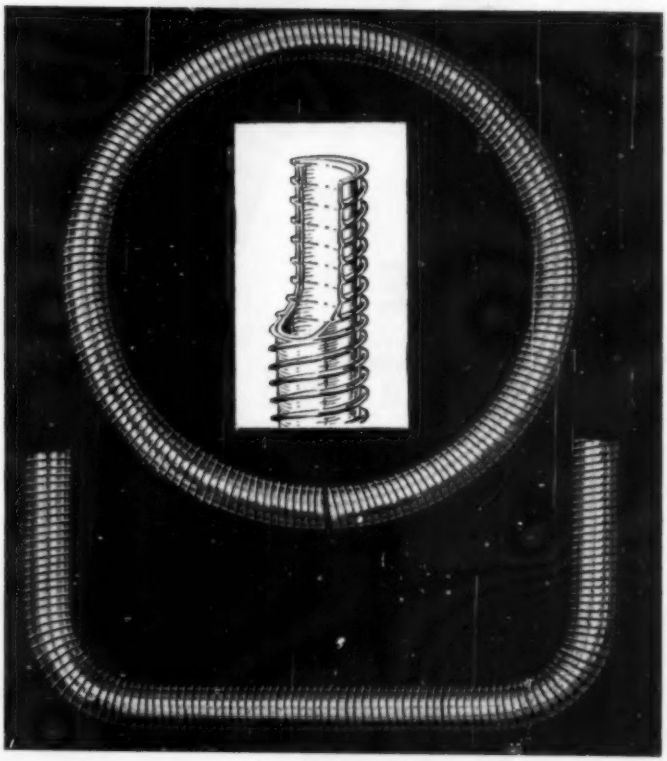

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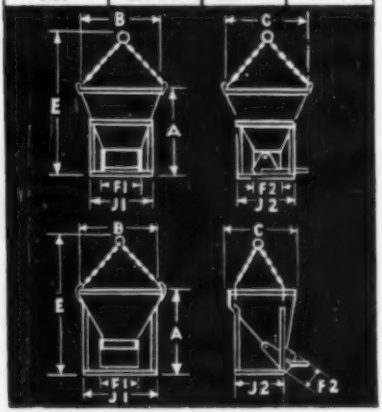

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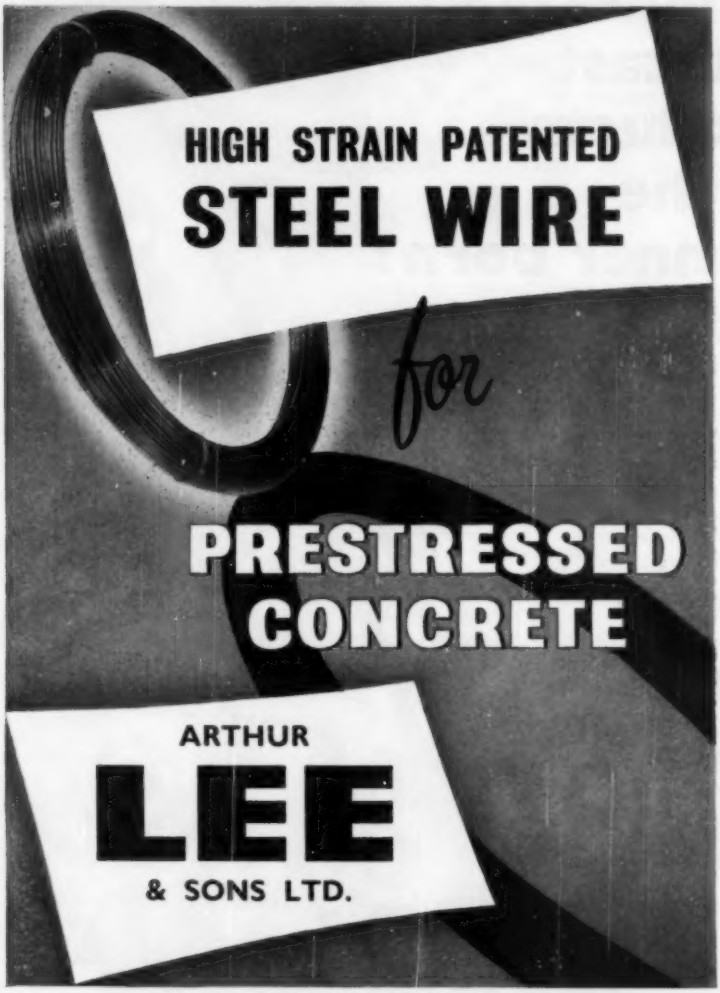

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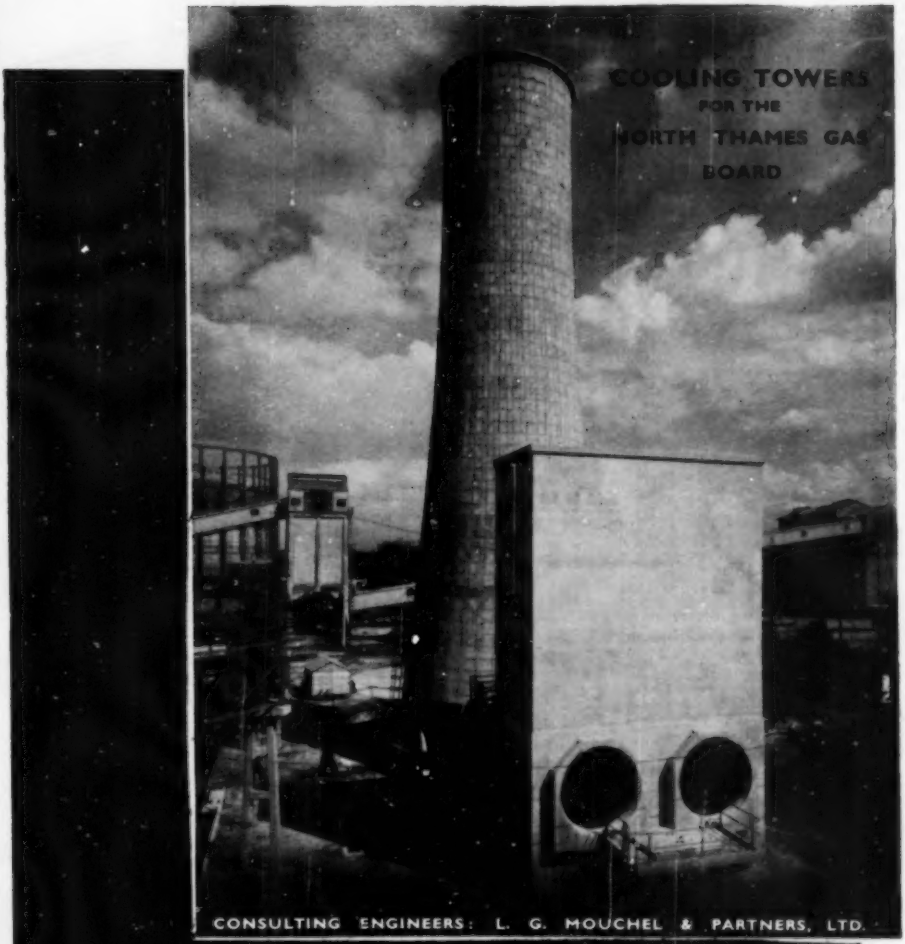

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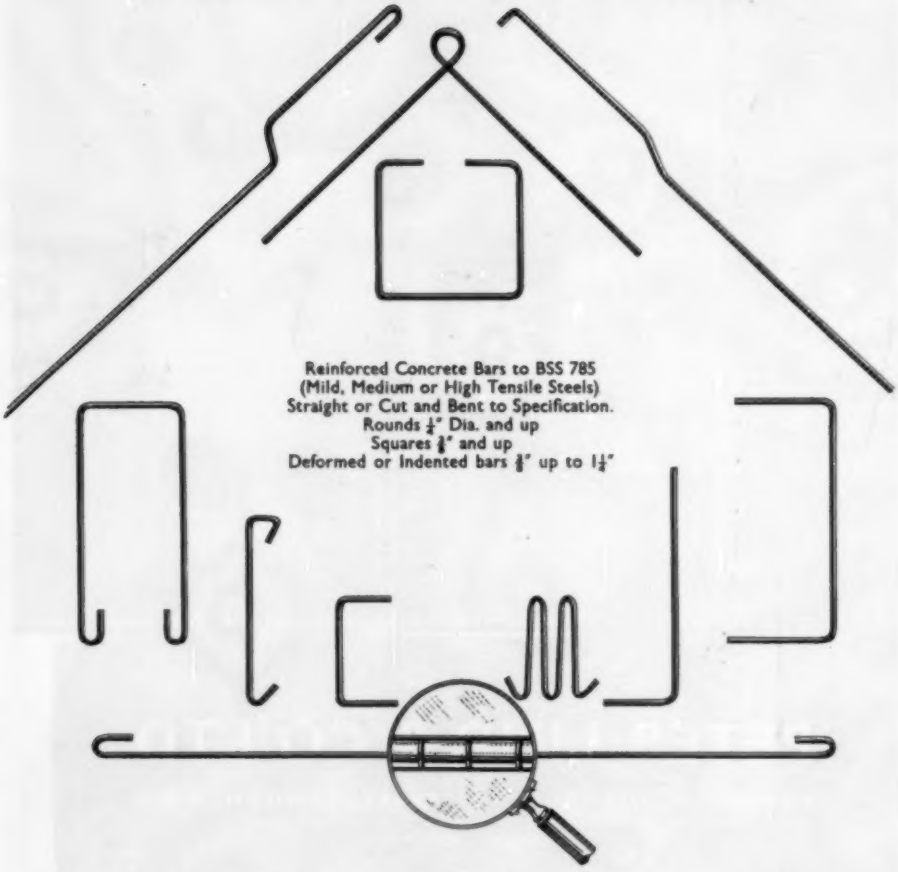

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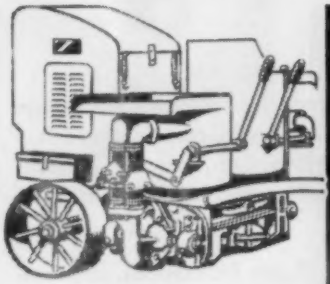

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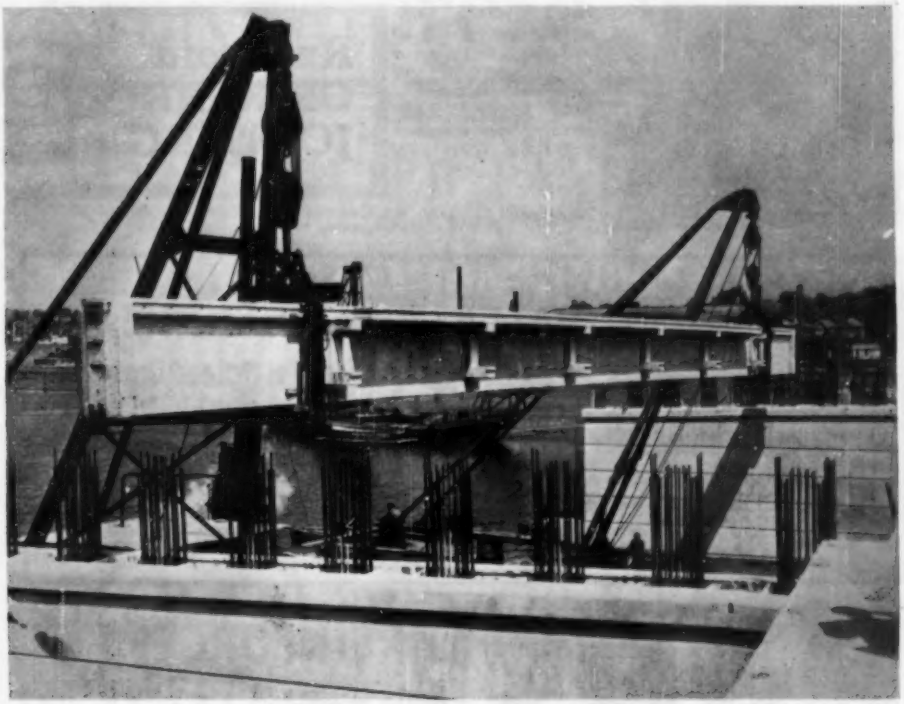

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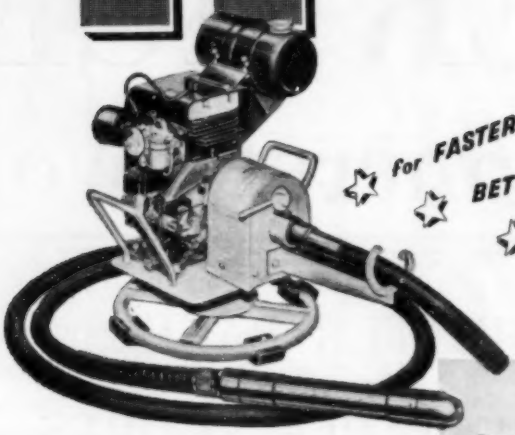

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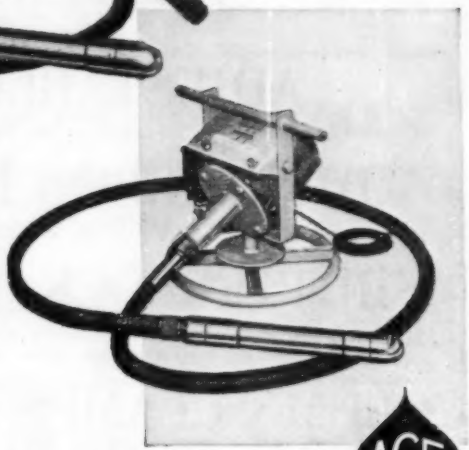

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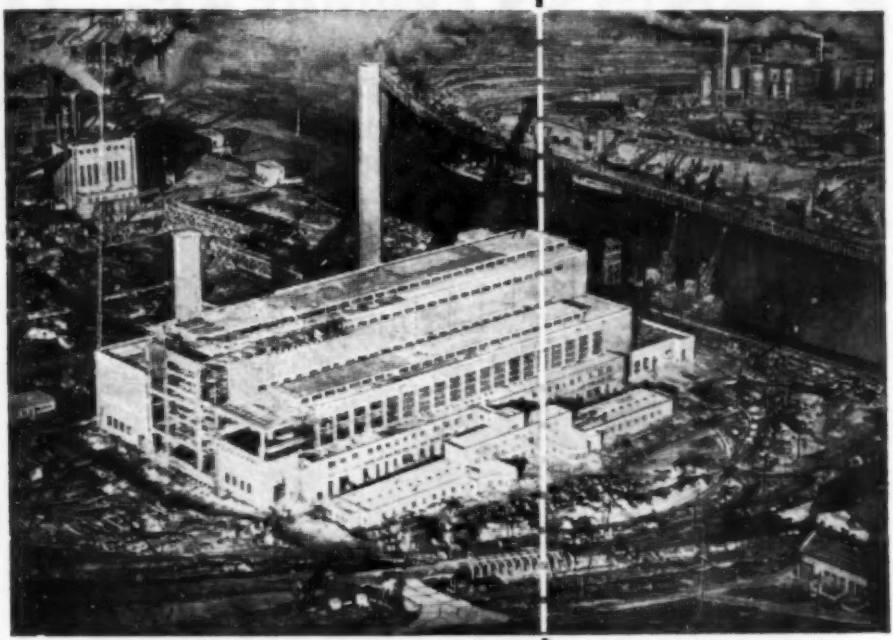

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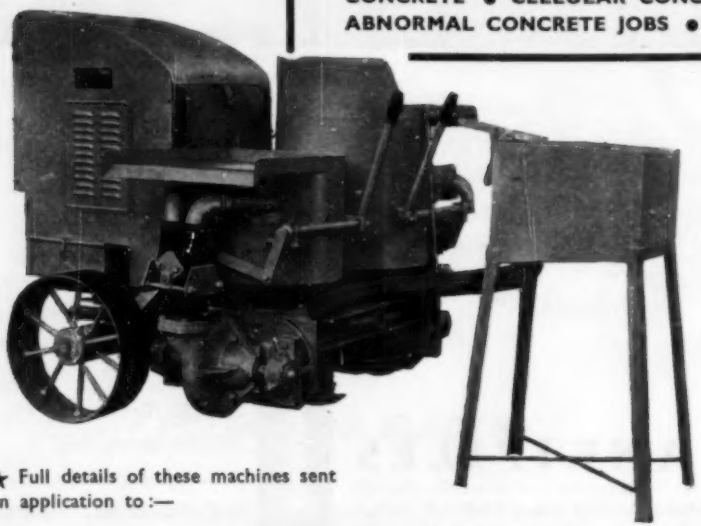

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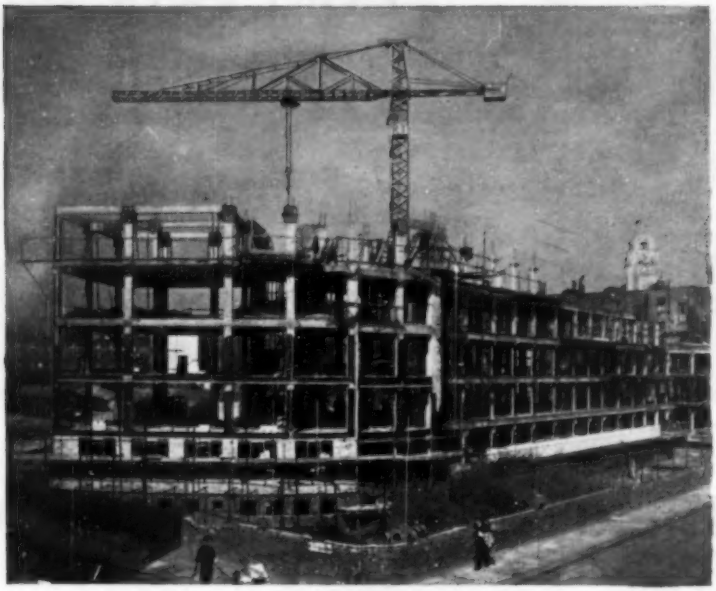

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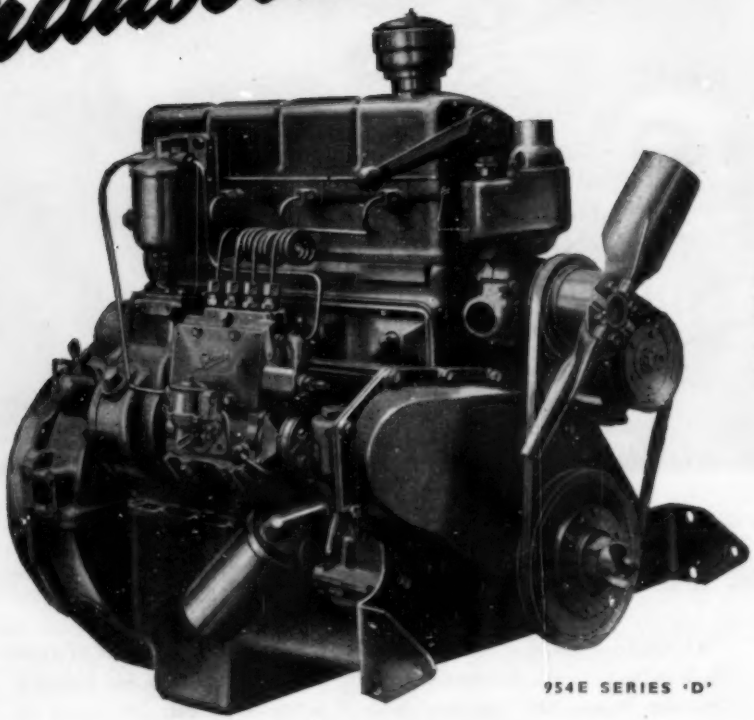

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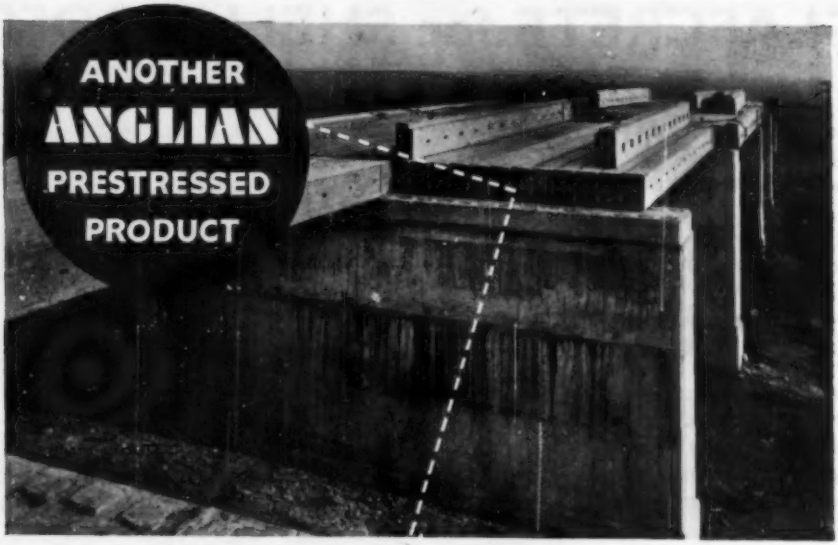

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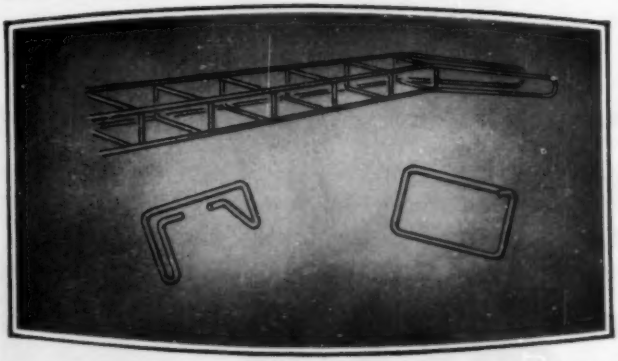

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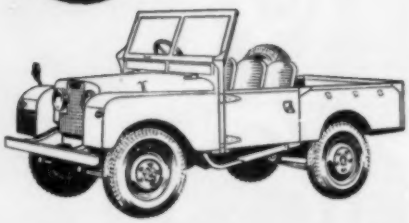

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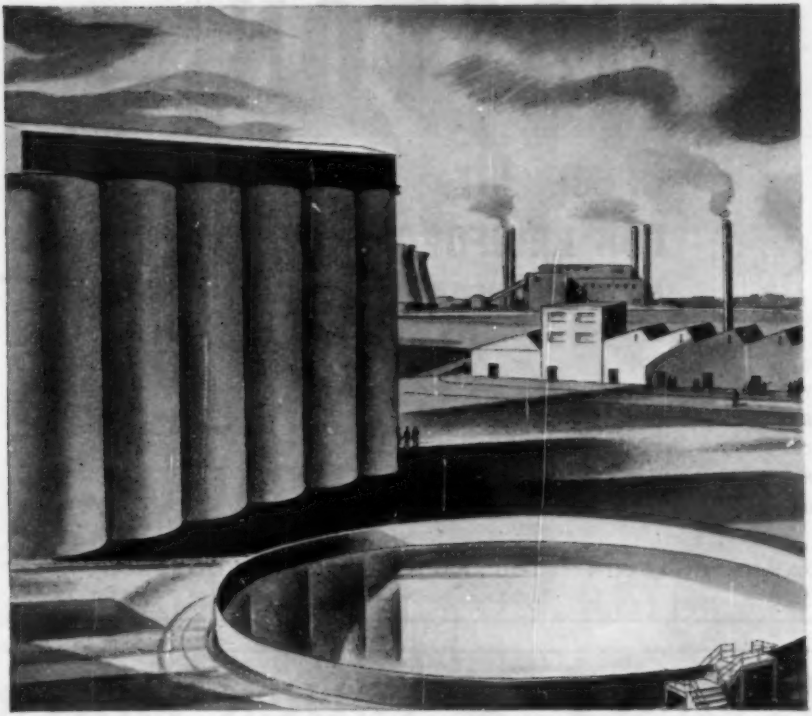

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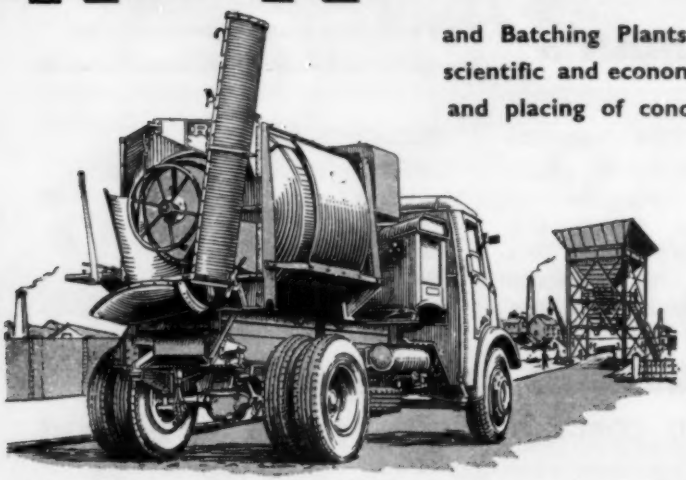

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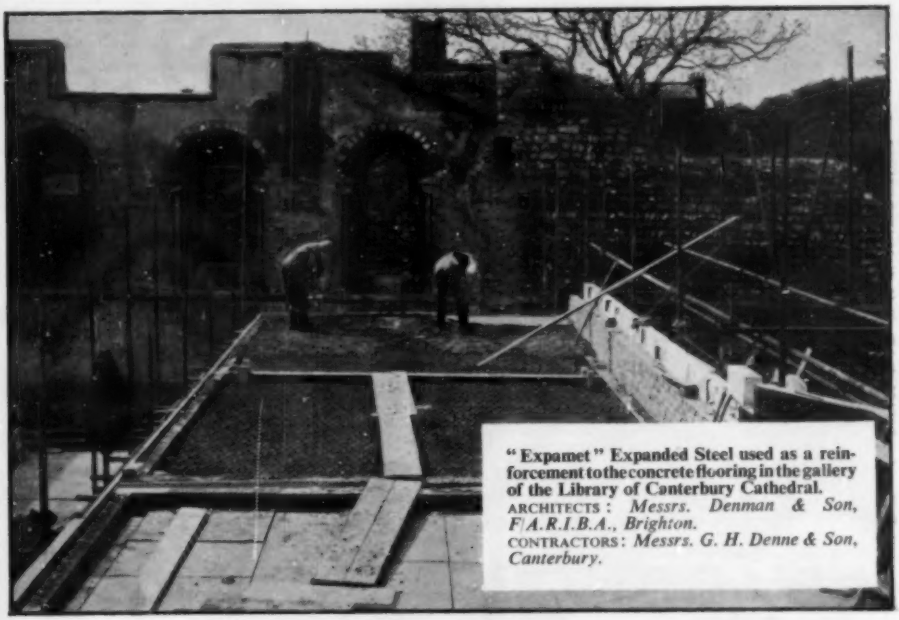

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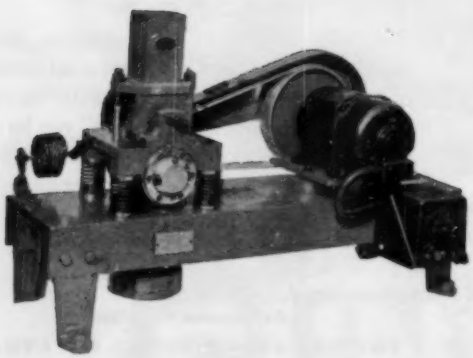

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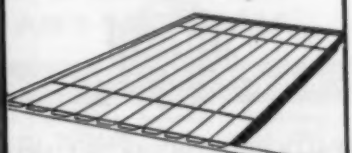

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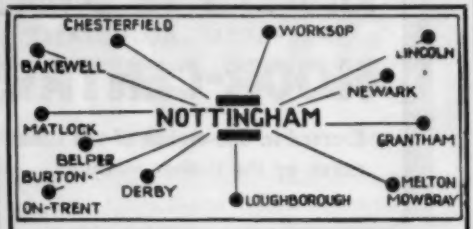

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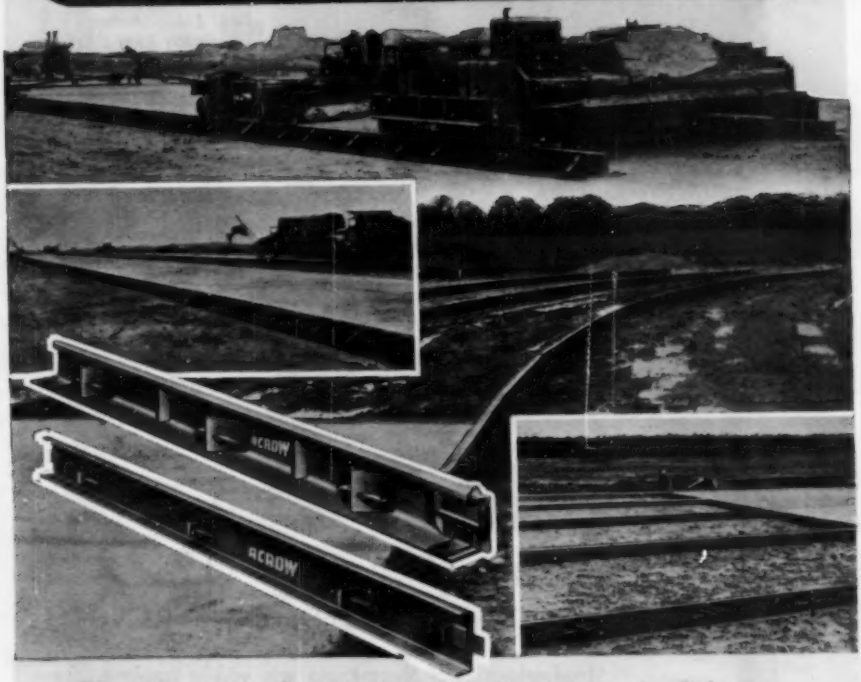

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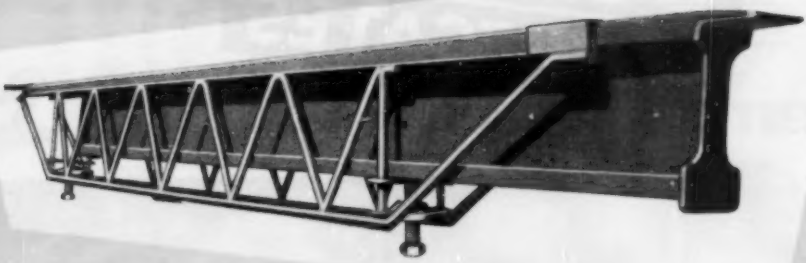

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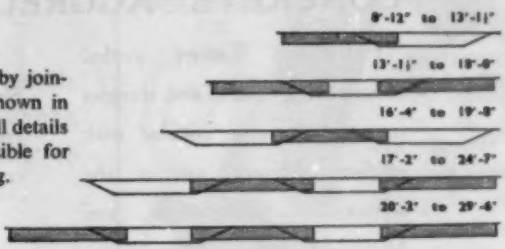

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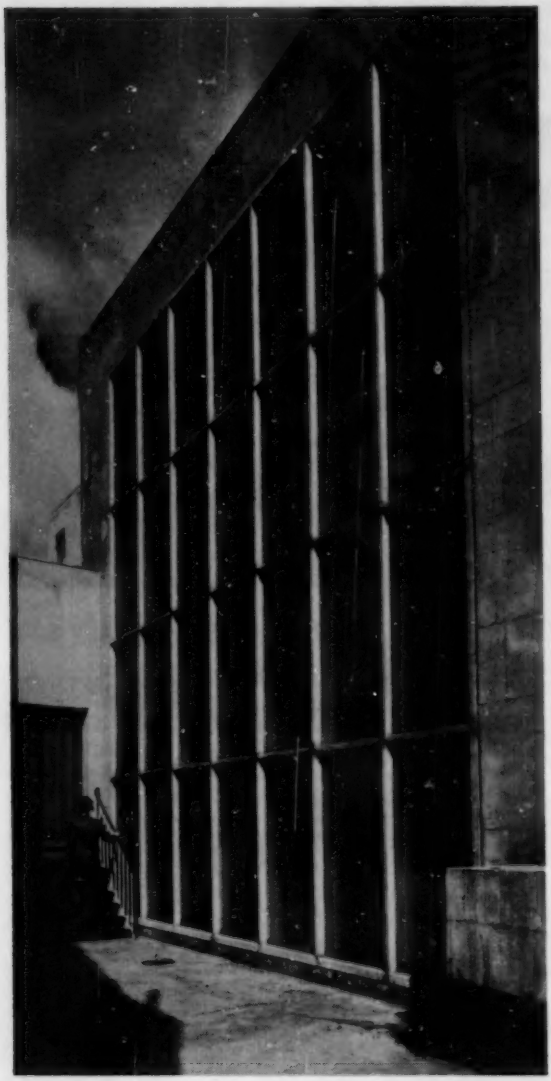

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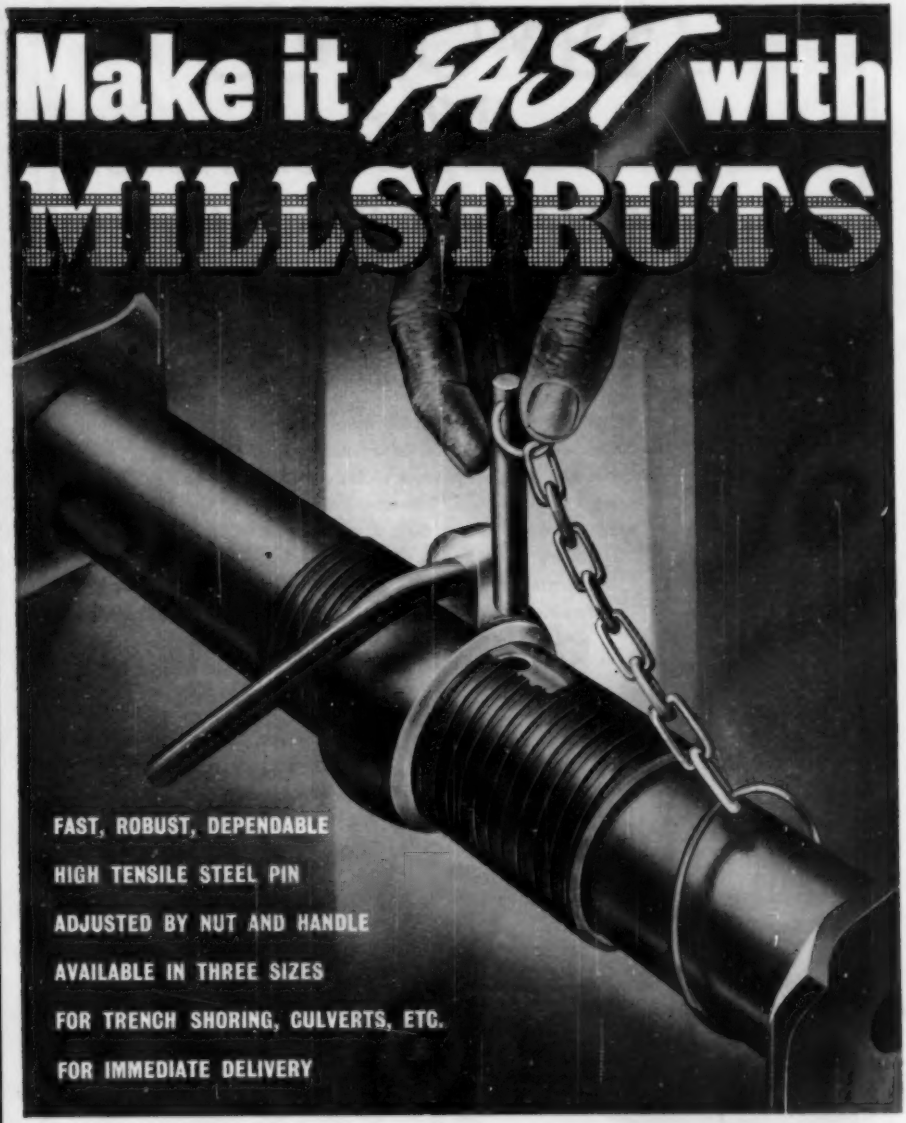
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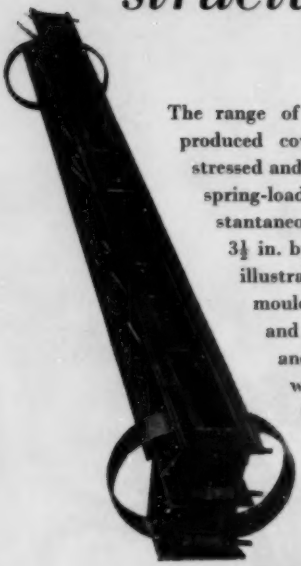

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Volume XLIX, No. 5. LONDON, MAY, 1954.

EDITORIAL NOTES

The Drafting of British Standards.

THE British Standards Institution has been granted a Royal Charter. The documents it issues are called British Standards. It might be thought that documents so entitled and issued by such a body would be carefully drafted and say what they mean. Indeed there should be no possibility of doubt about a document which many public authorities adopt in place of a specification. These documents were formerly called Specifications, and it was probably because some of them were so loosely worded and had permissive clauses, and were consequently in no sense specifications, that they are now called Standards. One recently issued British Standard is indeed so badly drafted that it should be withdrawn and issued again after careful revision if it is still thought that it is really necessary.

This standard, No. 892, "Glossary of Highway Engineering Terms", is dated 1940 but was issued only a few months ago. The reason for its publication is stated to be that development during the past twenty years has "increased the vocabulary of terms and demands a more strictly scientific terminology." Some of the terms recommended are so ridiculous and against common usage that they will never be adopted. For example, it is recommended that the word soil be used to describe only the loose top layer of the ground that supports vegetation. It is quite wrong to limit the use of the generic term soil in this way. This layer of soil has long been known as top soil, and will continue to be so known in spite of this new "British Standard" definition. The senselessness of the new definition is seen later in the definition of the term sub-soil, which is described as "the undisturbed strata lying immediately below the top soil". The use of the plural strata instead of the singular stratum is inexcusable, but it is also seen that the compilers themselves find it necessary to use the term top soil to describe what they recommend should be described as soil. The definition of bridle roads does not allow for their use by equestrians, which is their primary purpose. A footpath becomes a fieldpath when it crosses fields or similar enclosed lands, and there is no word to describe it when it passes through woodlands. A causeway must always be paved. The place where spoil is deposited is called a bank, irrespective of whether or not the material is tipped into a disused pit or other depression. The verb to haul becomes a noun describing distance. This attempt at a "strictly scientific terminology" permits the use of the word dyke to describe a large ditch, a large earthen bank, or a dry stone wall. A flagged

or bricked pavement is called a paving, and the word pavement is defined as the whole superficial area of a road. In the next section, however, the part reserved for pedestrians becomes a foot pavement and vehicles will use a carriageway. A concrete road becomes a cement-concrete pavement. The desire to force upon us the American term pavement to describe a carriageway has confused the compilers to such an extent that they have been driven to absurd terms such as " road pavement " and " foot pavement ". There seems also to be confusion and redundancy in the use of the recommended terms Formation, Base, Sub-base, Foundation, Road-bed, and Sub-crust. The common terms rolling and vibrating are discarded in favour of the one word consolidating, so that if this glossary were adopted it would be necessary to say consolidating by rolling instead of simply rolling ; there seems no advantage in this cumbersomeness, particularly as the word ramming is permitted to describe the operation of consolidating with a rammer. The word blinding is limited to the application of a layer of loose material to fill voids or interstices ; the use of concrete for this purpose is not mentioned. An expansion joint is said to be designed to permit small relative movement under thermal changes without destroying continuity, whereas the destruction of continuity is the sole purpose of such a joint.

Another recently-issued Standard is No. 2028, " Precast Concrete Blocks ", which is intended to define the recommended properties of blocks made with lightweight and dense aggregates. By a bad arrangement of a table it may be thought that blocks 18 in. by 9 in. by 9 in. only must be hollow if made with dense aggregate, and consequently that all the smaller sizes must be solid. We understand that this ambiguity has already been noticed and that a correction will be inserted in later copies. It is required that no block shall exceed 75 lb. in weight, which means that the density of a nominal 18-in. by 9-in. by 6-in. solid block made with gravel aggregate for use in load-bearing walls may not exceed 137 lb. per cubic foot. A footnote states that, if blocks are required with greater compressive strengths than those recommended by the Standard, such blocks should comply with the other recommendations, but no guidance is given on how extra strength is to be obtained without exceeding the greatest weight permitted. In the foreword it is said that the Standard will enable the characteristics of each type of product to be fully developed, but this is hardly consistent with the low limit of density required for blocks for load-bearing walls made with gravel or other dense aggregates. A footnote mentions that blocks 24 in. by 12 in. are permitted but not recommended. The density of a solid block of this size only 4 in. thick for a load-bearing wall and made with dense aggregate must not exceed 112½ lb. per cubic foot if it is to comply with the requirement that no block shall exceed 75 lb. in weight.

The Institution attempts to put the sole responsibility for these Standards upon the committees that compile them. But it is the Institution that issues them and confers upon them their high-sounding titles. It seems to us that the Institution has a duty to consider these documents carefully before they are issued, and to ensure that there are no mistakes and that they contain no ambiguities. Some of the recommendations in the Standard on highway terms are certain to be flouted, as some of them are by its compilers. In the Standard on concrete blocks it would have been better to have limited the maximum weight of 75 lb. to blocks made with lightweight aggregate.

Tests of Prestressed Precast Concrete.

By P. W. ABELES, D.Sc., M.I.Struct.E.

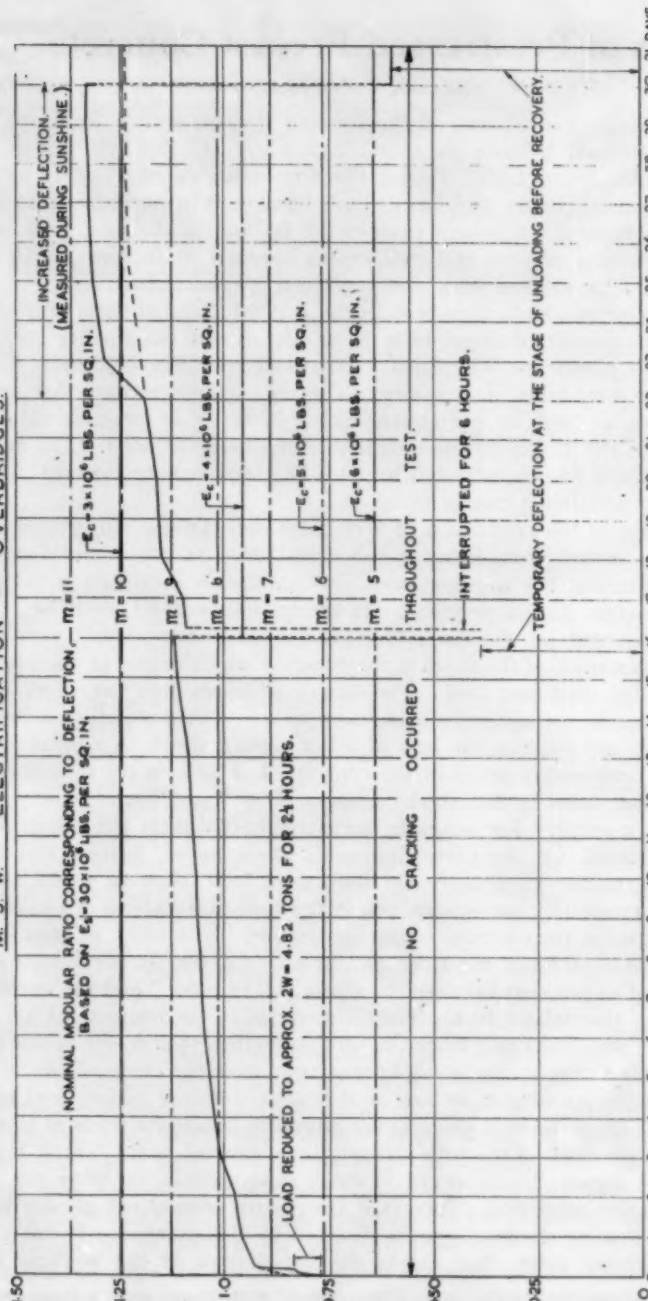
THE Civil Engineer's Department of the Eastern Region, British Railways, has been using prestressed concrete for more than five years, the first bridge contracts having been let in 1948. During this time some twenty bridges as well as roofs with purlins, girders, and beams have been built in partially-prestressed composite construction. Members between 12 ft. and 50 ft. long were made by the pre-tensioning process, and roof beams between 78 ft. and 102 ft. long were precast and prestressed with post-tensioned Magnel cables.

Routine acceptance tests have been made on nearly 3000 of these prestressed members. The purpose of these tests is to investigate the quality and the efficiency of the prestress. The quality is assessed by the deflection, which indicates Young's modulus of elasticity. The requirement is that this should not be less than 4×10^6 lb. per square inch; however, in practice values of 5×10^6 to $6{\cdot}5 \times 10^6$ lb. per square inch have often been obtained. The magnitude of the prestress was investigated by a loading approaching the load causing cracks, without permitting cracks to occur.

The efficiency of the prestress is of very great importance. In general practice, in most cases full prestressing is considered (that is when tensile stresses must not occur under the working load), but sometimes little care is taken to determine the actual loss of prestress. In many cases general losses of 15 per cent. only are assumed, but the magnitude of these losses depends to a very great extent on the magnitude of the stress in the concrete which occurs in the plane of the centroid of the tensioned steel. The sum of all losses may vary very much and may be 50,000 lb. per square inch and even more. Obviously, the percentage of losses depends not only on the sum of all the losses, which to a great extent depend on the compressive stress in the concrete, but also on the magnitude of the initial tensile stress in the steel. Consequently losses much greater than 15 per cent. may occur. For example, measurements during the construction of a bridge designed by the Civil Engineer's Department, British Railways, Eastern Region, in the years 1948 and 1949 show that, with an initial tensile stress of about 150,000 lb. per square inch in the steel and a stress of 140,000 lb. per square inch when the concrete is first compressed (the stress " at transfer "), the effective tensile stress in the steel may be only 110,000 lb. per square inch, which is a loss of 22 per cent between the stress at " transfer " and the minimum effective stress; this relates to a compressive stress in the concrete at transfer of about 2000 lb. per square inch when the cube strength is 5500 lb. per square inch.

Dr. Glanville * has found in his fatigue tests that the creep of concrete is proportional to the stress only so long as the stress does not exceed 0·33 to 0·4 of its strength. On account of this it is not advisable to allow a stress at transfer greater than 40 per cent. of the cube strength, as suggested in the " First Report on Prestressed Concrete ". However, in many cases stresses up to 50 per cent. and even more are permitted. Increased creep with consequent greater losses will occur, and thus no resultant gain is obtained by this practice. It was ascertained, in individual cases, that cracks developed even at the working load,

* W. H. Glanville, Building Research Technical Paper No. 12 (1930). W. H. Glanville and F. G. Thomas, Building Research Technical Paper No. 21 (1939).


M. S. W. ELECTRIFICATION — OVERBRIDGES.
NOMINAL MODULAR RATIO CORRESPONDING TO DEFLECTION BASED ON $E_c = 30 \times 10^6$ LBS. PER SQ. IN.
m = 11
m = 10
m = 9
m = 8
m = 7
m = 6
m = 5
$E_c = 3 \times 10^6$ LBS. PER SQ. IN.
$E_c = 4 \times 10^6$ LBS. PER SQ. IN.
$E_c = 5 \times 10^6$ LBS. PER SQ. IN.
$E_c = 6 \times 10^6$ LBS. PER SQ. IN.
INCREASED DEFLECTION (MEASURED DURING SUNSHINE.)
LOAD REDUCED TO APPROX. 2W = 4.82 TONS FOR 24 HOURS.
NO CRACKING OCCURRED THROUGHOUT TEST.
INTERRUPTED FOR 6 HOURS.
TEMPORARY DEFLECTION AT THE STAGE OF UNLOADING BEFORE RECOVERY.
DEFLECTION IN INCHES
1·50
1·25
1·0
0·75
0·50
0·25
0
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 DAYS
W W
4'-0"
26'-6"
2'-0"
2 WEBS
15 WEBS
BEAM D1

DEFLECTION OF BEAM DURING SUSTAINED LOADING OF 2W = 5·9 TONS.
EQUIVALENT TO WORKING LOAD PLUS 50 PER CENT OF LIVE LOAD CORRESPONDING TO A CONCRETE TENSILE STRESS OF 775 LBS. PER SQ. IN.

Fig. 1.

which was intended to correspond to zero stress in the tensile zone. In fact, owing to much greater losses occurring than had been anticipated, a stress occurred which equalled, or exceeded, the stress causing cracks.

Tests carried out by British Railways and others have proved that in pre-tensioned members the stress causing cracks, calculated for a homogeneous section,

Fig. 2.

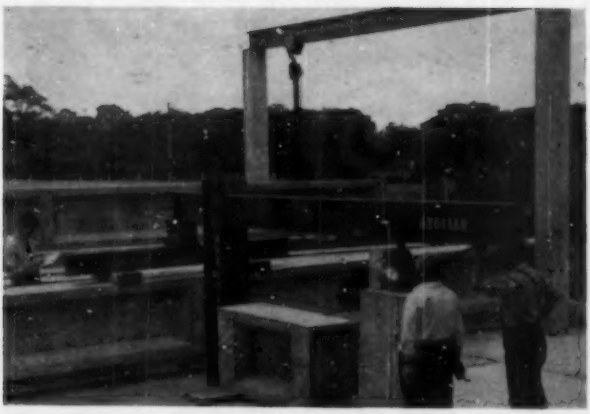


Fig. 3.

is about 1000 lb. per square inch for concrete with a 28-days' cube strength of about 7000 lb. per square inch. Based on this, numerous tests have been made in which the test load corresponded to a tensile stress in the concrete of 750 lb. to 800 lb. per square inch, considering the magnitude of the prestress at the time of testing. When these tests were first made early in 1949 it was thought by some that the requirement was too severe, but nearly all the tests between the years 1949 and 1952 were successful when one beam of each kind made on the

same production line was tested. Thus, it can be assumed that the conditions for the tests were quite reasonable.

In the first test at the works of Dowmac, Ltd., in 1949, sleepers were used as kentledge. In one case the load was sustained for thirty days, during which time the deflection increased (as can be seen from *Fig.* 1) without the development of cracks. *Fig.* 2 shows a test at the works of the Atlas Stone Co., Ltd., on a bridge beam 50 ft. long. *Fig.* 3 shows a test at the works of Anglian Building Products, Ltd., on an eaves beam 40 ft. long used in a roof at Bury St. Edmunds; this beam carries a central concentrated load from a roof beam 78 ft. long. In

Fig. 4.

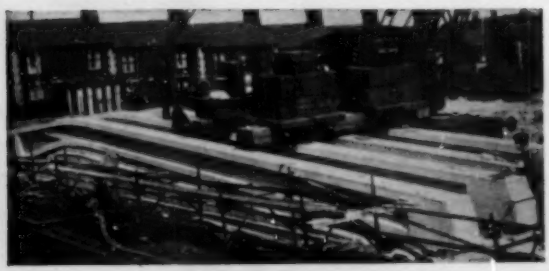


Fig. 5.

all these pre-tensioned beams the load corresponded to a tensile stress of 750 lb. to 800 lb. per square inch in the concrete. The permissible tensile stress under working load for highway bridges designed as partially prestressed was at first 550 lb. per square inch, which was later increased to 650 lb. per square inch after successful fatigue tests in 1952. For roof beams, in which no repetition of loading occurs as with bridges, 750 lb. per square inch has been allowed.

Routine loadings have been applied successfully also to precast beams with post-tensioned cables. In these cases the test load was reduced to a value corresponding to a tensile stress of 650 lb. per square inch in the concrete, since post-tensioned cables are more concentrated and are farther away from the tensile zone; as a result, cracks may occur at a slightly lower stress.

Fig. 4 shows the testing of beams for the Victoria Station roof, Sheffield,

described in the January, 1954, number of this journal. The load corresponded to a tensile stress in the concrete of 650 lb. per square inch, which equals the stress at the permissible working load. *Fig.* 5 shows the loading of a beam 102 ft. long for a loco. shed at Ipswich, for which the same stress of 650 lb. per square inch for the routine tests and permissible working load applied.

Cracks developed only in a very few instances out of the hundreds of tests made since 1949, and there was always a special cause for them which was noted and avoided in subsequent manufacture. Such causes were, for example, the occurrence of shrinkage cracks before "transfer," tensioning of the steel (in error) to a lower stress than that specified, and the use of aggregates containing a small proportion of chalk with consequently reduced tensile strength. These routine tests are considered to be essential for ascertaining the efficiency of the products. Moreover, they have proved that it is possible to apply partial prestressing, with its greater economy and increased resilience, as practised by the Eastern Region, British Railways, and still ensure freedom from cracks.

The author expresses his gratitude to Mr. J. I. Campbell, Civil Engineer, Eastern Region, British Railways, for permission to report on these tests.

Railway Goods Shed with Prestressed Beams.

THE goods shed illustrated is at Bury St. Edmunds, and is 200 ft. long by 78 ft. wide. The columns are of in-situ reinforced concrete and the beams are of prestressed concrete cast on the site and hoisted into position by two railway cranes. The work was designed by the staff of the Civil Engineer, Eastern Region, British Railways. Mr. Charles R. Price was the main contractor, and Anglian Building Products, Ltd., were sub-contractors for the prestressed work.

A Circular Diagram for the Design of Beams.

We have received from Mr. J. S. Savona, A.M.I.Struct.E., the following description of a graphical method of determining the depth of a beam subjected to a known bending moment and of calculating the stresses in a singly-reinforced beam.

The diagrams are related to the following equations in which M is the bending moment, b the breadth of the beam, d the effective depth of the beam, c the allowable compressive stress in the concrete, a_1 the ratio of the lever arm to the effective depth, and n_1 the ratio of the distance between the top of the beam and the neutral axis to the effective depth.

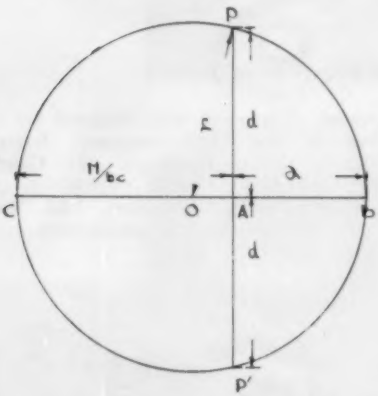


Fig. 1.

If $\frac{M}{bcd^2} = \frac{a_1 n_1}{2}$ and $\alpha = \frac{2}{a_1 n_1}$,

then $\frac{M}{bcd} = \frac{d}{\alpha}$.

To construct a diagram (*Fig.* 1), draw a horizontal line AB so that, to a suitable scale, BA = α; extend BA beyond A to C, making AC = $\frac{M}{bc}$ to the same scale. Bisect BC at O and, with centre O and radius OB or OC, draw a circle. Through A draw PAP′ perpendicular to BC to meet the circle at P and at P′, then AP = AP′ = d, that is the required depth of beam.

A geometrical proof of the construction is as follows.

$$AC : AP' = AP : \alpha,$$

but as $AC = \frac{M}{bc}$, and $\alpha = \frac{2}{a_1 n_1}$,

$$\frac{M}{bc} : AP' = AP : \frac{2}{a_1 n_1}.$$

Or, as $$\frac{M}{bc} : d = d : \frac{2}{a_1 n_1},$$

then $$AP = AP' = d.$$

In *Figs.* 3 and 4 values of α are given for values of n_1 and mp respectively,

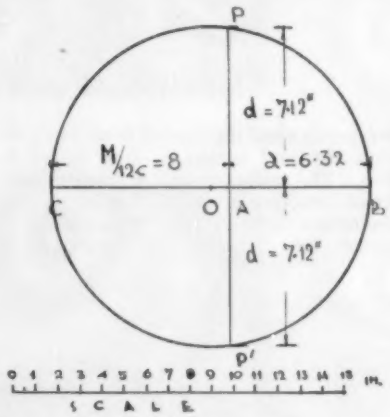


Fig. 2.

where m is the modular ratio and p is the area of reinforcement expressed as a percentage of bd (the effective area of the beam).

Examples.

(I) Determine the effective depth of a slab to resist a bending moment of 72,000 in.-lb. if the allowable stresses are 750 lb. per square inch in the concrete and 27,000 lb. per square inch in the steel. $m = 20$.

If $r = \frac{t}{c}$, then $n_1 = \frac{m}{(m + r)} = \frac{20}{56}$ = 0·357.

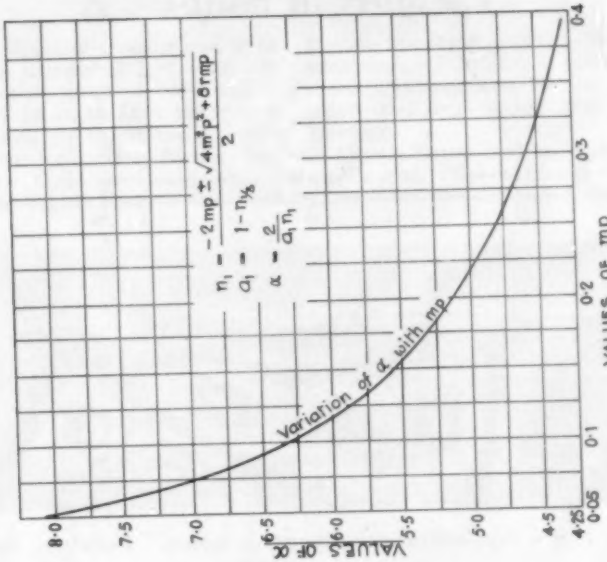

$n_1 = \frac{-2mp \pm \sqrt{4m^2p^2 + 8rmp}}{2}$
$a_1 = 1 - n_1^2/3$
$\alpha = \frac{2}{a_1 n_1}$
Variation of α with mp
VALUES OF α
VALUES OF mp.
8·0
7·5
7·0
6·5
6·0
5·5
5·0
4·5
4·25
0·05
0·1
0·2
0·3
0·4

Fig. 4.

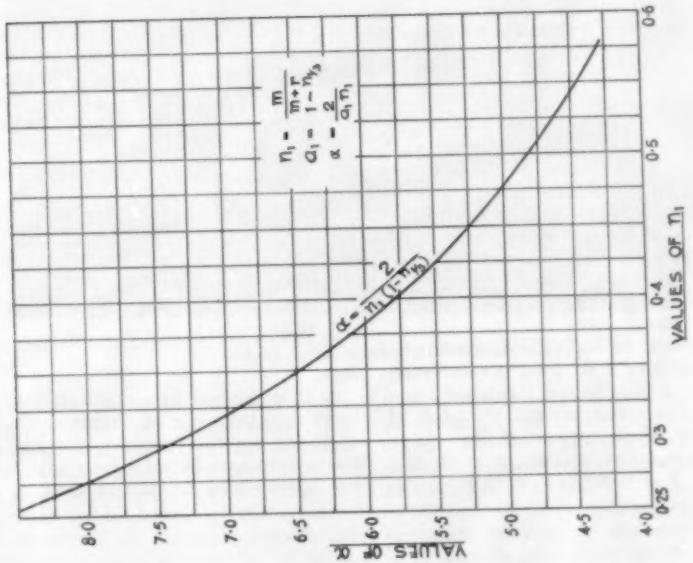

$n_1 = \frac{m}{m+r}$
$a_1 = 1 - n_1^2/3$
$\alpha = \frac{2}{a_1 n_1}$
$\alpha = \frac{2}{n_1(1-n_1^2/3)}$
VALUES OF α
VALUES OF n_1
8·0
7·5
7·0
6·5
6·0
5·5
5·0
4·5
4·0
0·25
0·3
0·4
0·5
0·6

Fig. 3.

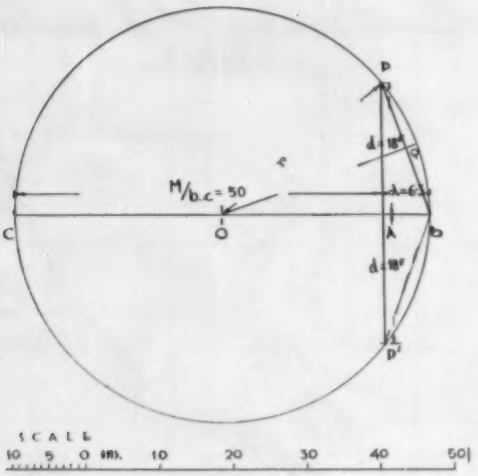


Fig. 5.

From *Fig.* 1, $\alpha = 6{\cdot}32$ and

$$\frac{M}{bc} = \frac{72{,}000}{12 \times 750} = 8.$$

Draw a horizontal line BA (*Fig.* 2) so that its length, to some convenient scale, is 6·32 units ($= \alpha$). Produce BA beyond A to C, making $AC = \frac{M}{bc}$ = a length of 8 units. Bisect BC at O and, with the centre at O, describe a circle through OB and OC. Through A draw PAP′ perpendicular to BC; then $PA = P'A = d = 7{\cdot}12$ in. to the same scale.

With the previously-determined value of α the value of mp is obtained from *Fig.* 2; thus $mp = 0{\cdot}1$ and $p = 0{\cdot}005$, and the area of reinforcement $= A_t = p.b.d = 0{\cdot}005 \times 12 \times 7{\cdot}12 = 0{\cdot}4272$ sq. in. per foot.

(II). A beam 12 in. wide and with an effective depth of 1 ft. 6 in. is reinforced with two $\frac{7}{8}$-in. bars. Calculate the stresses in the concrete and the steel if m is 18 and M is 240,000 in.-lb.

With $A_t = 1{\cdot}206$ sq. in., $b = 12$ in., $d = 18$ in., $p = 0{\cdot}00558$; with $m = 18$, $mp = 0{\cdot}1005$. Therefore, from *Fig.* 2. $\alpha = 6{\cdot}3$.

Draw AB horizontally (*Fig.* 5), and equal in scale to 6·3 units ($= \alpha$). Through A draw PAP′ perpendicular to AB, making PA = AP′ = 18 in. to scale. Join P to B, and B to P′; bisect PB or BP′, and let Q be the point of bisection of BP. Through Q draw QO perpendicular to BP to meet BA produced to the left of the diagram at O. With the centre at O and the radius OB, draw a circle to cut BO produced beyond O at C. Then $AC = \frac{M}{bc} = 50$ units to scale. Therefore if $\frac{M}{bc} = 50$, $M = 240{,}000$ in.-lb. and $b = 12$ in.; then $c = 400$ lb. per square inch.

From *Fig.* 1, $n_1 = 0{\cdot}358$ and, as $t = \frac{mc(1 - n_1)}{n_1}$, $t = 12{,}900$ lb. per square inch.

If it is desired to use the diagrams for large members it is advisable to use units of ft.-lb. for the bending moments, feet for the cross-sectional dimensions, and lb. per square foot for the stresses.

A Stadium in Hungary.

THE People's Stadium (*Figs.* 1 and 2) in Budapest was completed last year. It covers an area of about 67 acres and is surrounded by an earth bank rising to a height of about 30 ft. which provides standing room for 25,000 people and 22,000 seats. There are also 25,000 seats on raised stands built of reinforced concrete. On one side are cloakrooms, dressing rooms and other rooms. A road 150 ft. wide leads from the stadium to an underground railway station. There is a water pool 550 ft. long by 30 ft. wide on each side of the road.

The stand is 48 ft. high, mainly of framed construction, divided by 18 staircase enclosures. The seats are on terraces, supported by reinforced concrete beams, spanning between the staircase enclosures. Provision is made for eight more staircases and for an eight-story office building above the cloakroom building.

All the beams and seats were precast on the site. The seats and steps on the earth bank are supported on reinforced concrete piles. The seats (*Figs.* 3 and 4) are L-shape, and covered with wooden slats. The seats are sloped so that rainwater is directed to a drain which extends around the stadium.

There are two staircases in each enclosure leading to the seats. *Fig.* 5 is a cross section through an enclosure. The foundation of an enclosure is just above the level of the ground-water level

Fig. 1.—Exterior View.

Fig. 2.—View of the Stands.

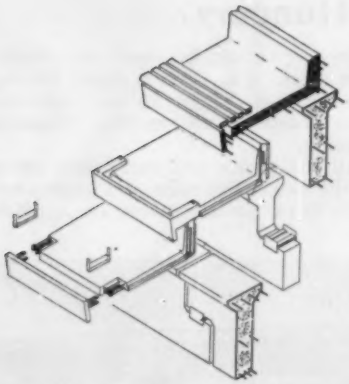


Fig. 3.—Details of Seats.

Fig. 4.—The Seats.

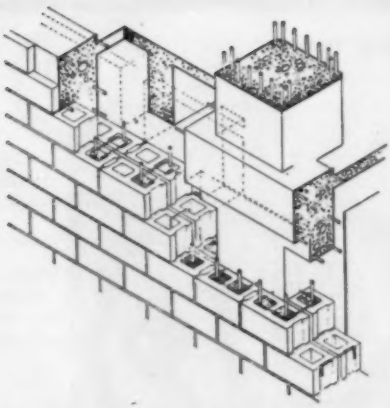


Fig. 6.—Walls of Hollow Blocks.

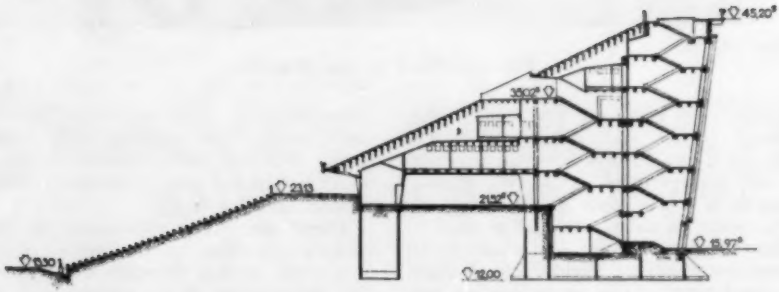


Fig. 5.—Cross Section through Enclosure.

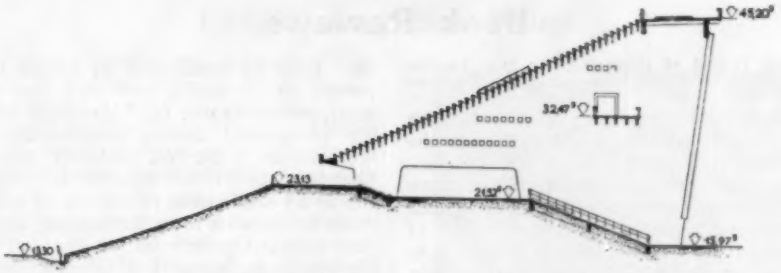


Fig. 7.—Cross Section through Terrace for Standing Spectators.

and comprises concrete slabs about 8 ft. wide. These slabs are connected to walls designed to resist wind forces. The walls of the enclosures are in-situ concrete to ground level, above which they are of hollow concrete blocks (*Fig.* 6), the cavities of some of the blocks being filled with concrete and reinforced. The walls are 150 ft. long and 108 ft. high.

The beams carrying the seats are designed as continuous beams with hinges a short distance from each support (*Fig.* 3). They are tee-shaped, and the thickness of flanges varies from 6 in. to 8 in. according to the span. The average depth of a beam is 2 ft. 8 in.; they are in lengths of 27 ft. to 40 ft. and weigh from 3 tons to 6 tons each. Ribs between the beams prevent lateral movement at the hinges.

The upper gallery between the staircases is supported by two main beams, 35 ft. long, carrying transverse beams which are arranged radially from the centre of the arena. A cross section of the terrace for standing spectators is given in *Fig.* 7.

Roof used as Movable Working Platform.

THE steel roof, weighing 110 tons, of the building shown in the illustration was used as a working platform and gradually raised as construction proceeded. Sliding shutters were used and 2600 cu. ft. of concrete were cast in nine days. The building, 90 ft. high, is at Persberg, Sweden, and the contractors were the Siab Co.

A Large Tower Crane.

A TALL tower crane has been imported from France by Messrs. Wates, Ltd., for use in the construction of blocks of eleven-story flats at Wandsworth, London. The crane is 300 ft. high at its fullest extent, and the radius of the jib is 78 ft. through a complete circle. Its lifting capacity is 3 tons at 23 ft., reducing to 15 cwt. at the end of the jib. The lifting speed for light loads is 200 ft. per minute. Movement of the crane is by three electric motors. The crane travels on a track of 12 ft. gauge.

Book Reviews.

"Field Testing of Concrete." By Sven Thaulow. (Oslo: Norsk Cementforening. 1952. Price not stated.)

THIS is an abridgement in the English language of a book by the author in the Norwegian language in which he describes the equipment and tests necessary for making good quality concrete. The equipment required comprises a set of standard sieves, a scale weighing up to 20 kg., a cylindrical container with a capacity of 10 litres, a slump cone, a plate of transparent material 10 in. square, a ⅝-in. rod of stainless steel 24 in. long, a small shovel, a hand-operated "drop-table" with an automatic counter for the number of blows given to the concrete, cylindrical steel moulds 6 in. by 12 in. and 4 in. by 8 in. and a portable compression-testing machine of 100-tons capacity. The object of the book is to describe simple and strong apparatus suitable for use on a site together with the tests which the author considers to be commonly necessary. Briefly these tests are to determine the grading and moisture content of the aggregates, the consistency, unit weight and composition of fresh concrete, and the compressive strength, weight, and water absorption of hardened concrete.

"Bautechnik-Archiv." Heft. 9. (Berlin: Wilhelm Ernst & Sohn. 1935. Price 14.50 D.M.)

THIS book contains the following essays. (1) "Tests on the stability of retaining walls subjected to earth pressure and stiffened by horizontal friction-slabs", by Heinz Muhs. The tests show that effective anchorage may be obtained by horizontal friction-slabs for walls without surcharge designed in accordance with Coulomb's theory even if the slab is in the wedge formed by the natural slope and plane of rupture of the material. (2) "The behaviour of piled foundations subjected to horizontal vibration", by Günther Brandes. In these tests horizontal dynamic forces were produced by vibrating machines and, even when the oscillation was constant and the soil poor, the piles vibrated freely to a limited depth only. The resistance to vibration of vertical piles was not appreciably less than that of inclined piles. To determine numerically the elastic yield of piles transversely to their axes it is suggested that they be considered as beams supported on an elastic bed with free and unsupported heads. (3) "Artificial freezing of ground during construction of foundations", by W. Sichardt and P. Chardabellas. Deals with the freezing of soil as an alternative to the use of cofferdams or caissons in water-bearing strata. The physical properties of the soil are discussed, including the penetration of frost into adjacent soil. Data are given on refrigeration capacities and plant required.

"Steel Reinforcement: Cutting, Bending and Fixing." By L. A. Disney. 78 pages. 100 illustrations. (London: Concrete Publications, Ltd. 1954. Price 6s.)

THIS book is intended for draughtsmen and detailers in preparing bending schedules for steel reinforcement, and also for builders and men on the site engaged in cutting, bending, and fixing steel in reinforced concrete structures. The office work is dealt with in information on ordering the steel, and examples of detailed drawings and bending schedules with notes on their preparation. For the man on the site there are full explanations of how to read working drawings, and chapters on practical methods of cutting, bending, and fixing the steel. An introductory chapter explains the properties of reinforced concrete, and should help to improve workmanship by giving the steel fixer a knowledge of the reasons why the drawings should be followed exactly.

"Beitrag zur Berechnung kreuzweise gespannter Fahrbahnplatten im Stahlbruckenbau." By Gerhard Fischer. (Berlin: Wilhelm Ernst & Sohn. 1952. Price 10 D.M.)

ISSUED on behalf of the German Steel Union, this booklet gives a method of calculation for slabs supported on four edges applicable to steel bridges. Instead of using the general solution of Navier's and Fourier's double series, the method employs a simple series resulting in a solution requiring much less arithmetic. For continuous slabs use is made of a method due to Bleich, and tables are provided giving values of the functions in the formulæ. The study includes two examples which do not, however, appear to be directly applicable to reinforced concrete design.

Large-diameter Piles.

We have received the following from Mr. L. E. Hunter, M.Sc., A.M.I.C.E.

A method of constructing foundations has been developed in France in which soil is removed by a grab from large-diameter tubes which are later filled with concrete, generally reinforced, and then generally withdrawn. This method is of value in very hard soil or boulders, or where the depth required is such that piling or open-well work would be uneconomical or impossible. Also if heavy concentrated loads have to be supported and a piled foundation of the usual kind is impracticable, a large pile of 4 ft. to 5 ft. diameter reduces the difficulties of supporting such loads. The method, known as the Benoto system, comprises patented hammer-grabs worked by a winch and operating inside a tube with a diameter of from 3 ft. to 5 ft. As the soil is excavated by the grab the tube sinks into the earth, movement being aided by a reciprocating arm fixed to the tube.

The hammer-grabs consist of a cylindrical barrel containing the mechanism for operating the grabs. Various kinds of heads may be used for different strata and they are easily detachable. The usual type of head has a cylindrical upper section attached to the barrel, and the grabs have four leaves which can be opened or closed by the mechanism in the barrel which is operated from the cab of the winch. The hammer-grabs are used to break the rock or soil, and by closing the head the spoil is retained when the grab is lifted to the surface. For hard or soft subsoil or rock such as sandstone the same head is used for breaking and lifting the material. The head for normal work is shown in *Fig.* 1. For harder material the head is provided with a sharper point and for very hard material a very heavy blunt head or a sharp-pointed tool up to 3 tons in weight is used to break the rock and is then replaced with the normal grab which picks up the spoil.

In the case of foundations in deep water a large-diameter tube enables the bottom to be inspected, and in a 5-ft. tube hand

Fig. 2.—Trepan used for very Hard Rock.

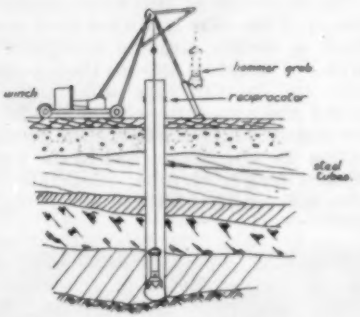


Fig. 3.—General Arrangement.

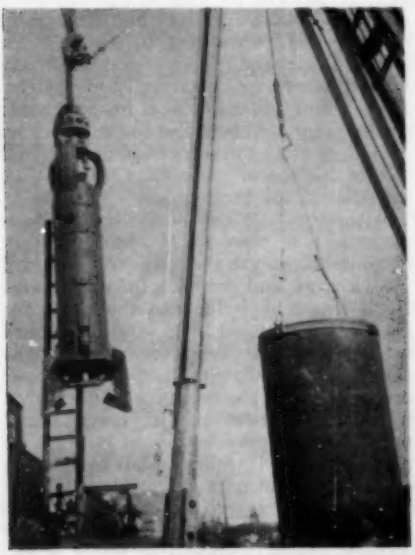


Fig. 1.—The Hammer Grab.

work is possible. The presence of water in the tube cushions the impact of the hammer and the weight of the hammer is reduced due to buoyancy; these two effects may seriously reduce the efficiency of the machine. For these conditions the head used has radial vanes connected to a circumferential guide-ring which allows water to flow through the vanes (*Fig.* 2); this head has a sharp point to break the rock. After sufficient rock is broken a normal grab is used to lift the material. If the boulders are spherical and tend to slip out of the jaws of the grab when they are being lifted, teeth may be fitted to the quadrants to increase their gripping power.

The reciprocating arm which assists the sinking of the tube is worked from a ram which is usually situated alongside the winch. The arm is fixed to the top of the tube, and as the tube sinks into the ground the arm falls with it. When a new length of tube is fixed to the previous portion the arm is attached to the top of the new length of tube. Generally the tubes are connected together in lengths

Fig. 6.—Hole ready for Sealing with Grouted Concrete.

Fig. 4.—Arrangement of Plant.

Fig. 5.—Grab with Teeth for Picking up Round Stones.

of 10 ft. to 13 ft. by welding the ends both inside and outside. The tubes are usually of mild steel about 0·4 in. thick. The grab is suspended from a tripod which can be adjusted for position and height. *Figs.* 3 and 4 show typical arrangements of the plant. Where the depth of boring is such that the tube cannot be removed due to friction of the subsoil, a new tube is required for each hole.

After the hole has been dug the reinforcement is placed. If the hole is of considerable depth the bars can be formed into a cage and lowered into the tube. Each length is welded to the next, the bars being clamped at ground level until the welding has been completed. The cage is then lowered and further sections are added until the reinforcement extends from top to bottom. In this manner reinforcement may be placed in holes up to 300 ft. deep. Spacers can be used to ensure that the bars are properly placed and that the required cover is obtained.

The tube is concreted in the usual manner unless considerable water is encountered, when the water has to be sealed

off while the steel tube is in position. To do this the bottom of the bore is filled with coarse aggregate which is grouted by a tube which penetrates to the bottom.

Where extremely hard rock has to be cut or where large boulders have to be removed, the heaviest hammer-grab may not break the material. In such cases explosives can be placed centrally in the bottom of the hole or around all or part of the perimeter of the tube. The former method usually requires about 1 lb. of explosive and when water is encountered in considerable depth the tube is frequently damaged. With smaller charges placed around the periphery the danger of the tube being damaged is lessened. After the charges are exploded the debris is removed by grabbing.

In marine work the tube is guided in position by braces from temporary trestles and the equipment is supported on a staging. On land where the area of the bottom of the pile is to be increased to reduce the bearing pressure the tube is lifted and the lower part of the hole enlarged by hand.

This method of construction is particularly suitable when the loads are very heavy as in the case of multiple-story buildings, where one pile may have to support up to 600 tons. In the case of the flats at Marseilles,* designed by M. le Corbusier, the load of 2,000 tons per pier

* See this journal for September, 1953.

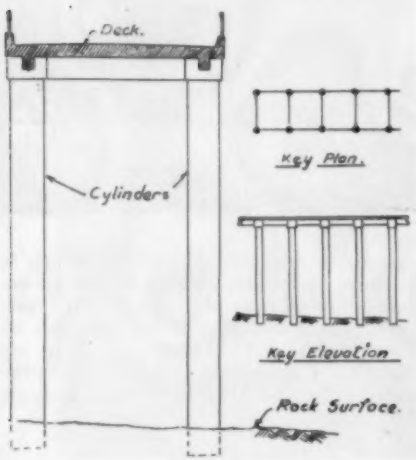


Fig. 7.

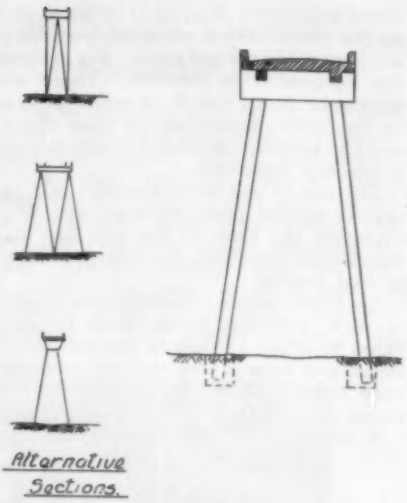


Fig. 8.

is supported by three 5-ft. diameter piles. The building is supported on marl at a depth of 40 ft. To increase their bearing capacity the piles were enlarged at the bottom by hand for a depth of about 8 ft. It is claimed that the average time required for boring for each of the 108 piles was fifteen hours. Generally, however, the loads to be supported are less than this and a tube smaller than 5 ft. diameter may be used for loads up to 400 tons.

At Stockholm the method is being used for the foundations of the Söder bridge over the Maeler lake. First it was necessary to stop movement of the bank of the lake, and this was done by tipping at the front of the bank 250,000 tons of stone through which were then driven caissons to glacial gravel to increase the shearing resistance of the stone embankment. The level of the top of the stone embankment is about 33 ft. below water level. In this case the tubes were 5 ft. diameter and were not withdrawn. Each hole was about 100 ft. deep and boring was difficult due to the presence of granite boulders and a sloping strata of solid granite. Explosives were used and about three weeks were required to excavate each hole. Three or four charges, each weighing about $\frac{1}{16}$ lb., were placed around the circumference of the bottom of the tube, and before they were exploded the tube was lifted about 5 ft. The grabs were

fitted with teeth (*Fig.* 5) in order to pick up the round stones which slipped out of the jaws of the normal grab. *Fig.* 6 shows the equipment in position. There was always 66 ft. to 100 ft. of water present in the holes, depending on their depth, and the bottom was sealed in the manner previously described.

The use of one large-diameter pile for several smaller piles decreases very considerably the area of the pile cap and accordingly simplifies the arrangement of ground beams.

For jetties where a foundation is desired in rock, sufficient depth must be provided to afford resistance against lateral forces, and for soft rock this will probably vary from 3 ft. to 6 ft. The vertical forces are not usually great, and a hole 5 ft. diameter will probably be sufficient in most cases. The tubes may be used as shutters and concreted to the level of the deck beams, which are connected to the reinforcement of the piles (*Fig.* 7). An alternative is to place precast columns in the holes while the tube is in position. The columns are grouted in the holes and, should raking members be required, can be pulled into position after the tubes have been removed (*Fig.* 8). The columns may also be prestressed to resist uplift.

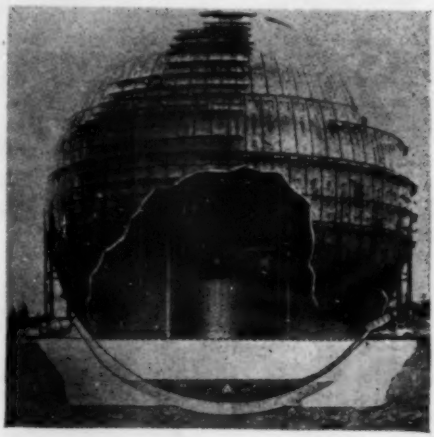
The method has also been used for the excavation of deep shafts, and one such shaft more than 300 ft. deep has been dug in this country in the Midlands.

The writer acknowledges the assistance given in the preparation of this article by Benoto, of Paris, Messrs. Braithwaite & Co., Ltd., and the Stockholm Harbour Board.

Foundation for a Sphere.

THE construction of the foundation of a sphere of 225 ft. diameter to contain an atomic reactor is described in " Engineering News-Record " for February, 1954. The sphere is built of steel 1 in. thick and weighs 3850 tons. It was supported over the excavation by 26 steel columns at its equator so that there was sufficient space under it to allow for inspection of the welds by means of X-rays and to ensure, by means of compressed air, that the joints were air-tight.

In order to avoid excessive stresses in the steel sphere, as the concrete was placed under the bottom crushed stone (A) was placed inside as shown in the illustration. The concrete under the sphere is 3 ft. thick, and the work was done by placing a layer of stone 3 ft. thick inside on one day and 3 ft. of concrete underneath the next day until both materials reached ground-level, which is 38 ft. above the bottom of the sphere. A total of 3900 cu. yd. of concrete and 28,000 tons of crushed stone were used. At ground level the sphere has a diameter of 179 ft. The concrete was placed through a chute arranged to travel around the sphere at ground-level. The



crushed stone inside the sphere, which is of 1-in. maximum size and was compacted to a relative density of 95 per cent., is covered with a concrete slab to form a floor. The work was done by the Rust Engineering Company, of Pittsburgh, for the U.S. Atomic Energy Commission.

A Prestressed Bridge in North Devon.

COMPOSITE PRECAST AND IN-SITU CONSTRUCTION.

ONE of the first bridges to be rebuilt in Devon after the floods of August, 1952, is the Martinhoe bridge, over the river Heddon at Hunters' Inn, which was completed in December last. The new bridge (*Fig.* 1) has a span of 40 ft. and is at an angle of 57 deg. to the river. It is designed to carry the Ministry of Transport's standard highway load with a small depth of construction to allow for a rise in the level of the river without having to raise the level of the road and bridge. The site precluded the use of full-length precast beams, and hollow trough-shaped members cast in short lengths were used. These were placed end to end on tubular scaffolding and prestressed to form beams spanning the river. The prestressing cables are kept in position by concrete diaphragms at intervals along the trough, and are anchored in solid precast blocks at the

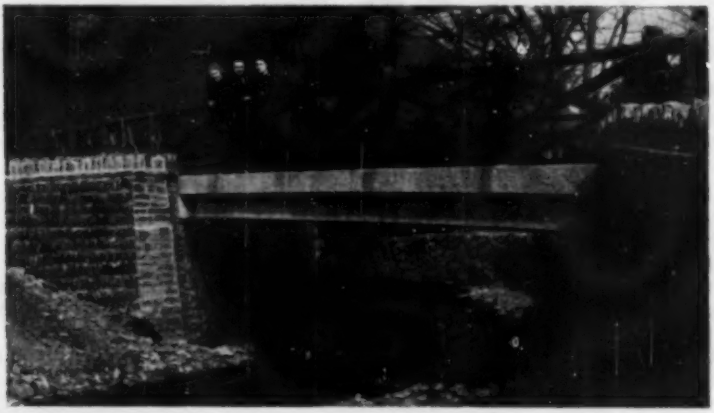


Fig. 1.

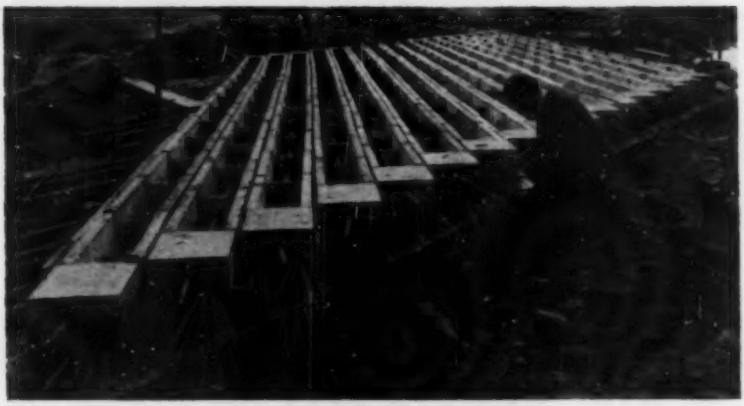


Fig. 2.—Tensioning the Main Cables.

ends of the beams. The hollow troughs were placed side by side to form the soffit of a slab (*Fig.* 2), and the bridge was completed by placing concrete in the troughs. The in-situ concrete was then vibrated to ensure bond with the prestressing cables.

Each row of troughs contains three cables, two of which are unsheathed and one encased in plastic sheathing. The unsheathed cables were tensioned before the in-situ concrete was placed and the sheathed cable was tensioned three days after concreting. Transverse prestressing was carried out by placing sheathed cables on top of the troughs before the slab was concreted. Special facing members were used to conceal the transverse anchor-cones and to carry the timber parapet.

The work was carried out in November and December, 1952, by six men who completed the deck in the equivalent of three weeks' working time. The cost of the completed deck, excluding the parapet and the surfacing, was 35*s.* per square foot.

The bridge was built for the Devon County Council, whose Surveyor is Mr. R. B. Carnegie, C.B.E. The consulting engineer for the contractors was Mr. E. W. H. Gifford, B.Sc., A.M.I.C.E.; the contractors were Messrs. J. J. Udall's Building Co., Ltd.

The bridge is prestressed by the Freyssinet system and the precast troughs, known as the "Udall Beam System", were made by the Liverpool Artificial Stone Co., Ltd.

The stone-faced plain concrete abutments and the timber parapet fence were designed and constructed by the Devon County Council.

A Vehicle for Loose Cement.

THE illustration shows a vehicle used in Germany for transporting loose cement. The tank, which is inclined towards the rear to assist in the discharging of the cement, is carried on a trailer. Inside the tank is a Polysius airslide formed by porous slabs made with a mixture of burnt silicates or clay. As the tank is cylindrical in cross section and the slabs are flat there is a small space between the bottom of the slabs and the tank in which any moisture which may be present in the tank will collect. Compressed air is supplied by a compressor driven by the engine of the towing vehicle, and the pressure is sufficient to eject the loose cement upwards to a tall silo. It is claimed that 13 tons of cement may be discharged in 8 to 15 minutes.

The tank may be divided into separate compartments so that several sites may be supplied by the same vehicle. The manufacturers (Haller G.m.b.H.) supply the vehicles with capacities of 12, 18, and 20 tons and a vehicle with a larger engine is available with a capacity of 25 tons.

A Container for Loose Cement.

Precast Concrete in School Buildings.

THE school shown in *Fig.* 1 consists of a light steel frame supporting precast concrete floor slabs and to which precast concrete wall slabs are connected. The components conform to a module of 3 ft. 4 in. The school shown in *Fig.* 2, for which the components are based on a module of 8 ft. 3 in., also consists of a light steel frame and precast floors and walls, but in this case the roof is of precast concrete slabs. The steel frames and the precast slabs are made by Messrs. Hills (West Bromwich), Ltd., and the method, which is suitable for other types of buildings, has been used in the construction of more than 300 schools.

THE LARGER SLABS.—The shapes of the slabs used for the module of 8 ft. 3 in. are shown in *Fig.* 3. The roof slabs are 8 ft. 2½ in. long by 1 ft. 3⅞ in. wide and weigh 270 lb. each. They are an inverted channel-shape in cross section, the depth of the ribs being 4 in. and of the top ⅞ in. The top is stiffened by transverse ribs at the ends and by three transverse intermediate ribs; at the bottom of all the transverse ribs are timber strips to which battens and plaster boards are fixed after erection of the slabs. The ribs are reinforced with mild steel bars and the slab with wire mesh.

The floor slabs have the same overall dimensions as the roof slabs but the ribs are 5 in. deep and the top 1½ in. thick. Usually the transverse ribs do not contain timber strips, but they can be

Fig. 1.—A School at Wokingham.

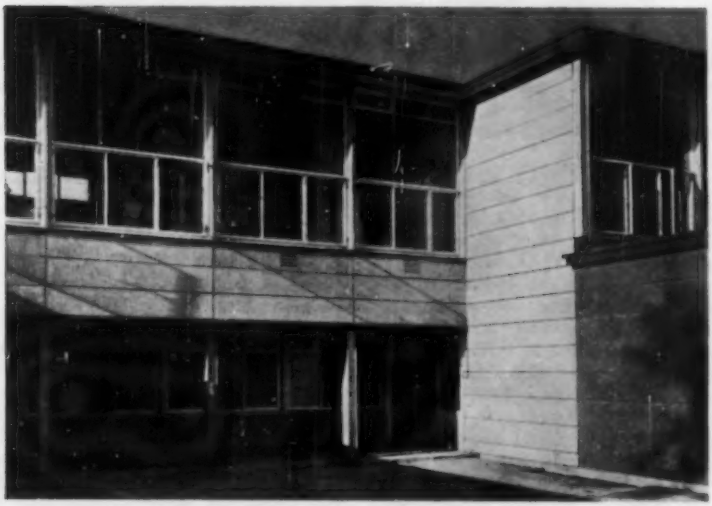


Fig. 2.—A School at Denmark Hill, London.

provided if required. Each slab weighs 350 lb. The reinforcement comprises mild steel bars in the ribs and wire mesh in the top.

The wall slabs are 8 ft. 2⅛ in. by 1 ft. 3⅛ in. and weigh 200 lb. A rib 1¾ in. wide by 2¼ in. deep is formed around the edges and the central part is 1¼ in. thick and stiffened by two transverse ribs. The slabs are reinforced with mild steel bars. They are laid with the longer dimensions horizontal; in the lower ribs a triangular groove is formed and in the upper rib there is a triangular tongue, so that the tongue of a lower slab keys in the groove in the slab above. The vertical edges have a semi-circular groove formed in them to act as a water-stop. In the upper rib of a slab as fixed there are rustproof ferrules, about 4 in. from each end, into which are fitted bolts which in turn are fixed by galvanised pressed-steel clips to the stanchions of the frame.

The wall, roof and floor slabs are of 1 : 1½ : 3 concrete with gravel aggregate. The concrete has a crushing strength at 28 days of at least 4,500 lb. per square inch.

The stanchions are at 8 ft. 3 in. centres, and, by means of the nut on the bolt fitting in the ferrule, the slabs are brought tightly against a packing of bituminous material and a sealing compound applied to the outer vertical face of a stanchion (*Fig.* 3). Open vertical joints are preferred to allow for expansion of buildings over 60 ft. long. In shorter buildings the vertical joints may be filled. The horizontal joints are ⅜ in. wide and are filled with mortar. The vertical joints are often left open, the entry of moisture being prevented by the damp-proof course behind.

The plinth of a wall is formed by precast members 2 ft. 8¾ in. long by 5⅛ in. by 4 in. The wall consists of an outer leaf of slabs, an air-space 1½ in. wide, and an inner leaf of lightweight blocks of any suitable material. The external faces of the slabs have a white or coloured exposed-aggregate finish. The facing-mixture is placed integrally with the slab.

THE SMALLER SLABS.—For buildings designed on a module of 3 ft. 4 in. the floor and wall components are shown in *Fig.* 4. The floor slabs are 3 ft. 3¾ in. long by 1 ft. 7⅞ in. wide. They are 1½ in. thick and stiffened by two longitudinal ribs 1¼ in. deep formed 2 1/16 in. from the

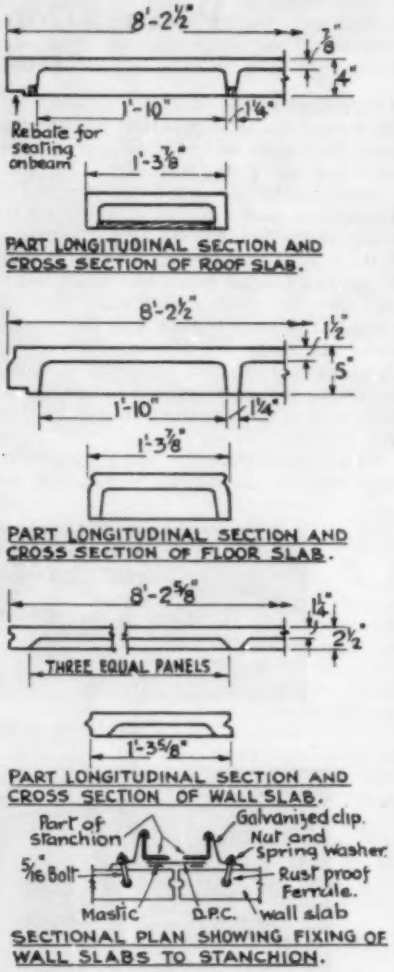


Fig. 3.—Components for Module of 8 ft. 3 in.

longer edges. The slabs have a roughened top surface to key with the finish, and the edges are shaped as shown in *Fig.* 4. The ribs are reinforced with mild steel bars and the tops with wire mesh. The gravel concrete is a 1 : 1½ : 3 mixture.

The walls have a single leaf only of slabs 3 ft. 3 3/16 in. long by 3¾ in. thick. The slabs for a top course are 1 ft. 7⅝ in. wide and the slabs for intermediate and bottom courses are 1 ft. 11⅝ in. wide (*Fig.* 4).

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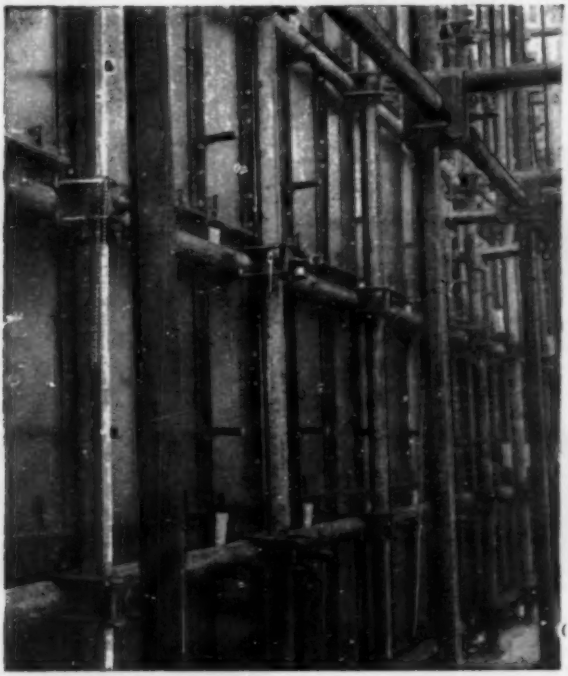

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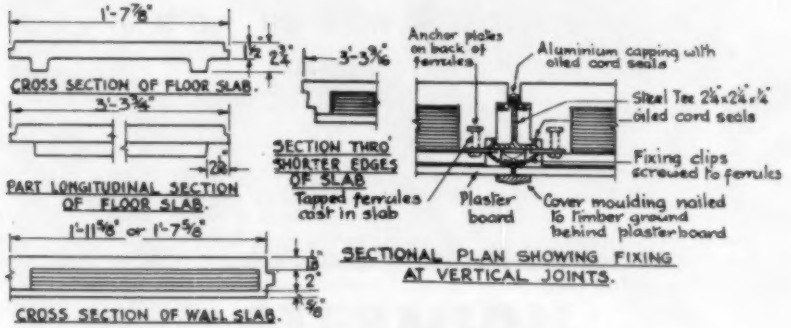
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Plaster board
Cover moulding nailed to timber ground behind plasterboard
SECTIONAL PLAN SHOWING FIXING AT VERTICAL JOINTS.
1'-11⅜" or 1'-7⅝"
CROSS SECTION OF WALL SLAB.

Fig. 4.—Components for Module of 3 ft. 4 in.

The slabs have a concrete face 1⅛ in. thick with upturned edges forming a recess 2 in. deep which is filled with concrete made with expanded-clay aggregate, or a woodwool slab; over the whole of the back of the slab is a sawdust-cement finish ⅜ in. thick. Rustproof fixing ferrules are cast in each slab near the corners.

These slabs are fixed to a framework of 2½ in. by 2½ in. by ¼ in. steel tees placed vertically at 3 ft. 4 in. centres and fastened to the steel frame. The method of attaching the slabs to the steel tees is shown in *Fig.* 4. The horizontal joints between the slabs are bedded at the back only with a 1 : 1 : 6 cement-lime-sand mortar, all the face joints being left open. At the intersections where four slabs abut are aluminium jointing pieces.

Fig. 5.—Erecting Face Slabs.

MANUFACTURE OF SLABS.—The upper surfaces of floor and roof slabs are finished with a trowel as the moulds are pushed along the production lines. Wall slabs are filled with gravel concrete to within about ½ in. of the top, and as they pass along the line a white or coloured sand-cement facing, to which a waterproofing compound has been added, is placed on top of the gravel concrete and the surface is levelled. The mortar is placed on the wet concrete from a small hopper, with a vibrator attached to it, which is clamped to the roller conveyor so that the mould may pass under the hopper. When the slab reaches the end of the conveyor, crushed spar or granite is sprinkled on the surface and rolled in.

In the manufacture of the smaller wall slabs a sawdust-cement layer ⅜ in. thick is spread on the bottom of the mould and a 2-in. woodwool slab placed on top. The dense concrete is then placed to within ¼ in. of the top of the mould and the facing mixture added as previously described. Fixing-ferrules and wooden nailing strips are placed in the moulds with the reinforcement.

The erection of slabs of the larger size is shown in *Fig.* 5.

The architect of the school shown in *Fig.* 1 is Mr. S. Johnson-Marshall, Chief Architect of the Ministry of Education, in collaboration with Mr. J. T. Castle, County Architect of Berkshire. The architect of the school in *Fig.* 2 is Dr. J. L. Martin, M.A., Architect of the London County Council.

Railway Bridge with an Arch of 300 ft. Span.

THE bridge (*Fig.* 1) over the river Fulda at Gunterhausen, Germany, replaces some of the arches of a masonry viaduct which were destroyed during the war. The undamaged arches on each side of the new portion are retained.

To enable the railway service to be maintained, the bridge was constructed in halves with a central longitudinal joint.

Fig. 1.

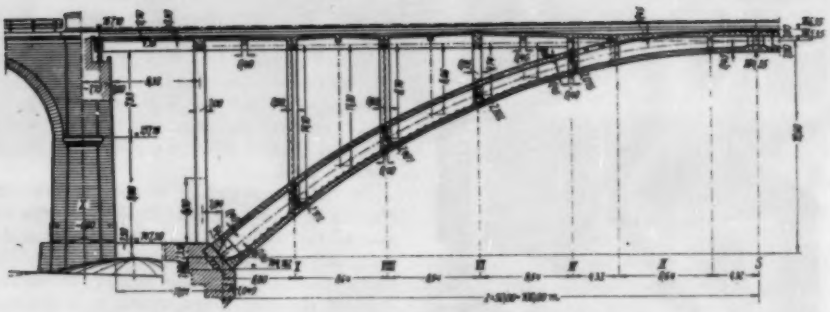

Fig. 2.—Part Longitudinal Section.

Each half comprises an arch with a span of about 300 ft. and a rise of about 60 ft. The arches have a hollow rectangular cross section and are 13 ft. wide by depths varying from 6 ft. 6 in. at the crown to 9 ft. 9 in. at the springings. The sides are 1 ft. 6 in. thick and the top and bottom are 1 ft. 6 in. thick at the crown increasing to 2 ft. near the springings. The deck in each longitudinal half is 15 ft. 6 in. wide and is supported on two longitudinal beams and on transverse beams at 14-ft. centres.

The longitudinal beams on each side of the central transverse joint are about 143 ft. long; each is a continuous beam on four intermediate supports. At the piers the beams are supported on reinforced concrete rockers, and the intermediate supports are columns at about 28-ft. centres carried on the arches. At the junctions with the arches the columns are hinged; the reinforcement for a hinge is shown in *Fig.* 4. The columns vary in size according to their height, the largest, at the springings, being about 3 ft. 3 in. square. A longitudinal section through the bridge and cross sections through one half of the deck and an arch are shown in *Figs.* 2 and 3 (the dimensions on these sections are in the metric system). On

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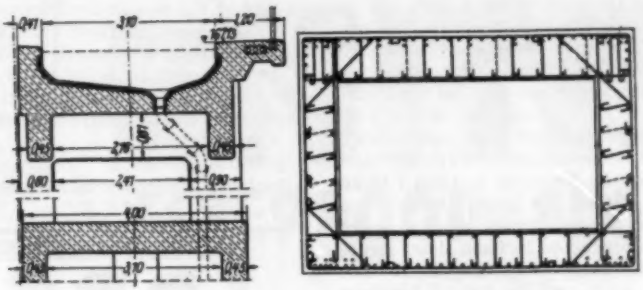

(*a*) (*b*)

Fig. 3.—(*a*) Cross Section of Deck and Part of Rib ; (*b*) Reinforcement of Rib.

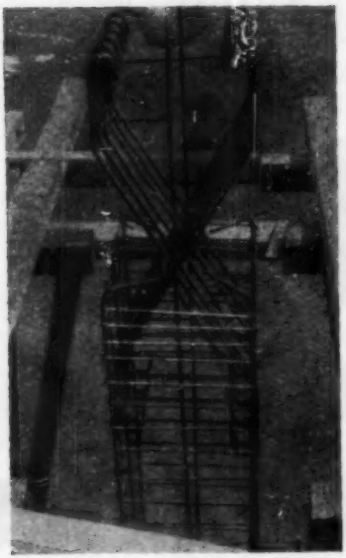

Fig. 4.—Hinge between Column and Rib.

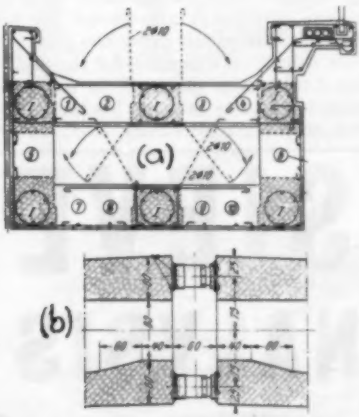

Fig. 5.—Joint at the Crown of the Arches. (*a*) Cross Section ; (*b*) Longitudinal Section.

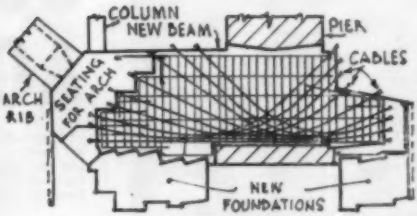

Fig. 6.—Foundation of Arch.

each side of the slab on each arch a kerb is formed to retain ballast under the railway track; one kerb on each slab also forms a footpath about 4 ft. wide.

The arches, which are fixed at the abutments, were cast with a gap of about 2 ft. at the crown, and ten hydraulic jacks, each of 200 tons capacity, were inserted in the gap in the positions marked 1 to 10 in *Fig.* 5. After the arches were compressed the shaded areas marked 1 in *Fig.* 5 were concreted to form short struts reinforced by loops projecting from each face of the joint and by helical binding. After these struts had hardened the jacks were removed and the gaps concreted.

The abutments of the masonry arches were strengthened to support the new structure. The masonry piers were retained and new concrete foundations were built on both sides of each pier (*Fig.* 6). The two new parts are joined by prestressed beams which carry the pier, and on one side form a bearing for the arches.

The bridge was designed to carry the standard German railway loading. The arches were considered as fixed at the ends, and the maximum horizontal thrust is about 1524 tons from each arch. The average crushing strength of the concrete at 28 days was 6600 lb. per square inch. The bridge was designed by the German Railways and the contractors were Wayss & Freytag, A.G.

[Abstracted from "Beton-und Stahlbetonbau," February and March, 1953, by L. J. Elgin, A.M.I.C.E.]

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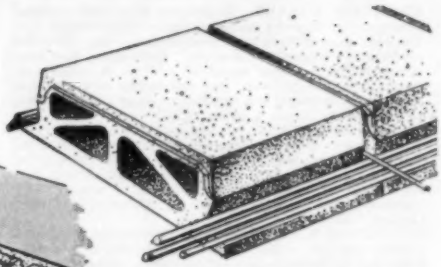
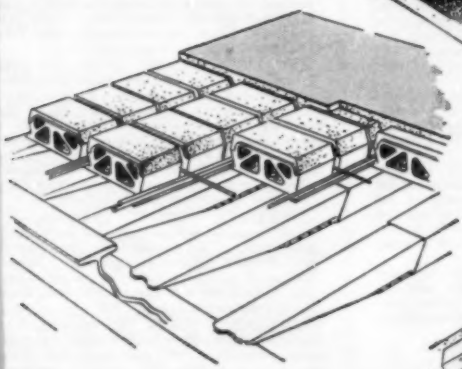


Showing Two-way Reinforcement and Hollow Concrete Blocks laid on Trianco Telescopic Centers.



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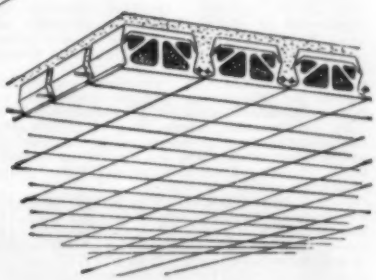

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(Continued on page 178.)

A Cantilevered Shell Roof.

THE shell roof shown in *Fig.* 3 is over the entrance to a house in Mexico City. It is a conoidal double canopy supported at the centres of its longer sides and cantilevering 16 ft. on both sides of the supports. The shell was designed and built by Sr. Felix Candela, who described it in the Journal of the American Concrete Institute for March, 1953.

The overall dimensions are about 40 ft. by 20 ft. with a thickness varying from $4\frac{3}{4}$ in. at the centre to $1\frac{3}{4}$ in. at the shorter sides, which are curved upwards to form small ribs (*Fig.* 1). An approximate analysis of the stresses was made by assuming the surface to be formed by four hyperbolic paraboloids with vertical axes, A'B'CD, ABCD, A'F'DE, and AFDE. This requires the substitution of two straight lines CD, DE, for the curve. According to M. Aimond * a characteristic of these surfaces is that the stress at every point is constant for loads parallel to the vertical axes of the surfaces and uniformly distributed over their horizontal projection. The double curvature has also a high resistance to

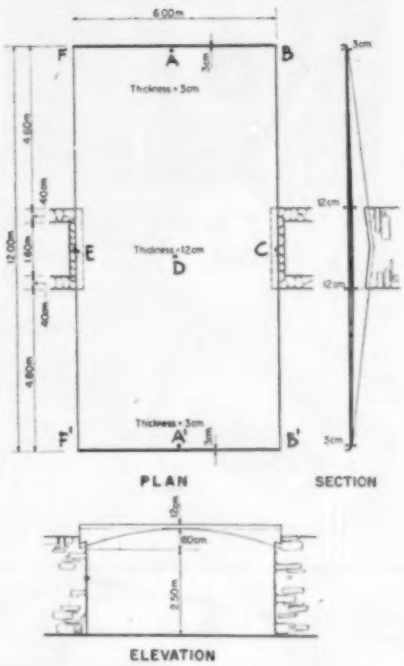

Fig. 1.—Dimensions of the Shell.

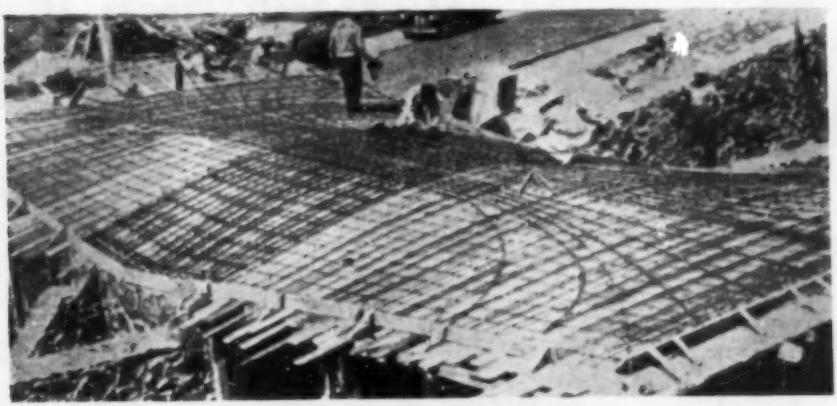

Fig. 2.—Arrangement of Reinforcement.

* "Etude Statique des Voiles Minces en Paraboloide Hyperbolique Travaillant sans Flexion," by F. Aimond (Proceedings of the International Association for Bridge and Structural Engineering, 1936).

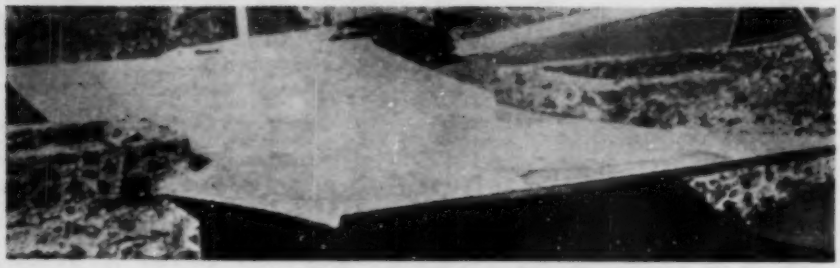

Fig. 3.—A Conoidal Double-canopy Shell Roof.

buckling. The central portion of the shell (about 8 ft. 3 in. wide) between the supports is designed as an arch and is reinforced near both the top and bottom. The remainder is reinforced with light steel mesh, extending over the whole slab at mid-depth, and with sixteen ½-in. bars on top of the mesh and curved on plan as shown in *Fig.* 2. The architect is Sr. Raul Fernandez.

MISCELLANEOUS ADVERTISEMENTS.
(*Continued from page lxx.*)

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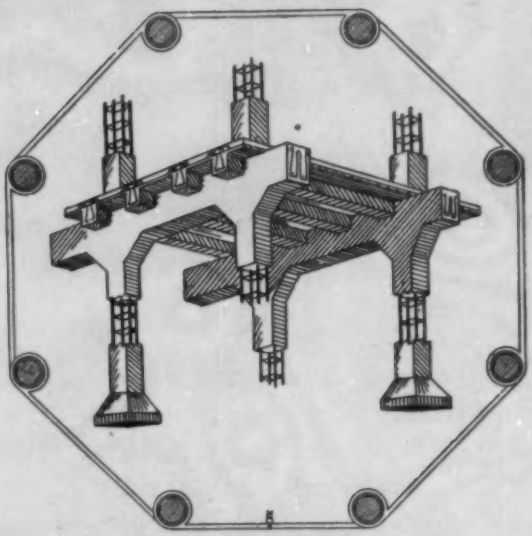

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